

**Static and Seismic Retrofitting of Benched Slopes
Explored through Centrifugal Model Tests and
Distinct Element Method**

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Abstract

The geometry of the slope surface significantly affects slope stability. Benching can substantially alter the slope configuration, transforming it from a simple straight line or curve into a series of steep stages, thereby enhancing stability. This study seeks to examine the failure mechanisms of benched slopes using centrifugal modeling and simulation, as well as to propose simplified methods to obtain their factor of safety. Additionally, an effective retrofitting method has been developed that can adapt to the variable conditions of construction sites.

A closed-form solution derived from the sliding wedge theory has been newly derived, enabling stability assessment of a uniform slope without iterative calculations. Additionally, the traditional two-dimensional sliding wedge method has been extended to three-dimensional conditions by incorporating lateral resistance, following retaining wall theory. Seismic conditions have been addressed by introducing a pseudo-static force to the sliding wedge. Based on these analytical methods for uniform slopes, the stability of benched slopes was evaluated, with comparisons drawn among different failure modes. Consequently, these derived theories have been utilized to forecast the failure of benched slopes observed through centrifugal model tests.

Subsequently, centrifugal model tests were performed to recognize factors influencing the stability of benched slopes and to observe realistic failure modes under different loading conditions. Boundary effects were also discussed. Through the installation of various sensors, deformation and variations in earth pressure within the model slopes were monitored. The measured data provided guidance for determining the values of lateral resistance. The experimental observations were used to validate the proposed theories. Theoretical predictions have shown good agreement with experimental results when water content variability is included.

Furthermore, three-dimensional distinct element models were constructed with reduced dimensions, adhering to the principles of centrifugal modeling and using scaled contact parameters. In addition to mutual verification with the experiments, these models helped explain unexpected phenomena observed during testing. Moreover, detailed failure patterns, unobservable in experiments, were captured. Compared to other simulation methods, this approach provides particle-level information, allowing for an in-depth examination of force chain evolution in benched slopes during loading and providing valuable insights into slope stability.

Finally, based on the identified weak areas in a benched slope, a seismic retrofitting method using shear pins was proposed. Various pin alignments were tested to gain a deeper understanding of their mechanisms under seismic loading.

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Chapter 1 Introduction

1.1 Background

Benching is an engineered slope retrofit technique that improves slope stability by removing material near the slope crest, thereby reducing the load on the slope (Hennes et al., 1959). This method reduces the overall slope angle by reshaping the terrain into terraced or stepped formations (Ghiassian et al., 2018). **Fig. 1-1** illustrates the configuration of a benched slope, where the dashed lines outline the original slope, and solid lines indicate the slope face after benching. The height of each bench is typically determined by the working height of the equipment used (Hantz, 1988).

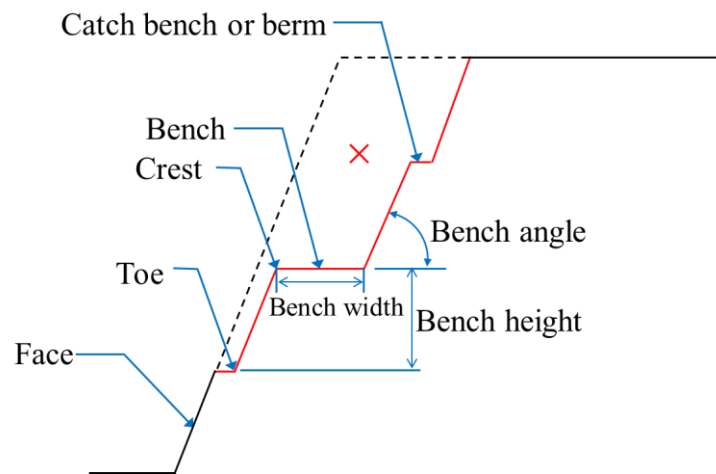


Fig. 1-1 Configuration of a benched slope

The stepped geometry of benched slopes not only enhances slope stability but also facilitates operational access, maintenance, and monitoring. This configuration can provide usable space on the bench platforms, supporting road and building construction in hilly areas. However, design flaws and poor management practices can compromise the stability of benched slopes. Additionally, natural disasters affecting benched slopes are occurring with increasing frequency worldwide, due to global climate change, leading to considerable loss of life and property.

On February 22, a major collapse happened at an open-pit mine in Inner Mongolia, China. Sliding soil from the benched pit wall buried personnel and equipment at the bottom of the site (see **Fig. 1-2 (a)**). This incident caused deaths of 53 people, 6 injuries, and an economic loss estimated at 200 million yuan (*Investigation report on the "February 22" major collapse accident at the open-pit coal mine of inner mongolia alashan xinjing coal industry co., ltd.*, 2023). The primary cause of the collapse was a failure to follow the original design specifications; specifically, benches were merged arbitrarily, creating excessively high and steep slopes. Additionally, continuous high-intensity stripping and coal extraction at the base of the working face further undermined slope stability.

Other accidents have occurred because of natural disasters. For instance, on May 1, 2024, a landslide

occurred on a road subgrade slope along the Mei–Da Expressway in Guangdong Province, China. This tragedy resulted in 48 deaths, 30 injuries, and over 20 vehicles falling down the mountain (Yue, 2024). The roadbed of this highway was constructed using the cut-and-bench-fill method (see **Fig. 1-2 (b)**), where the upper section of the original mountain surface was excavated to fill the lower section, thereby creating a benched slope with the expressway paved on the bench platform. The landslide originated in the fill section, beginning at the road centerline. According to the investigation, the primary causes of the incident were intense rainfall and the unfavorable material properties of the site.

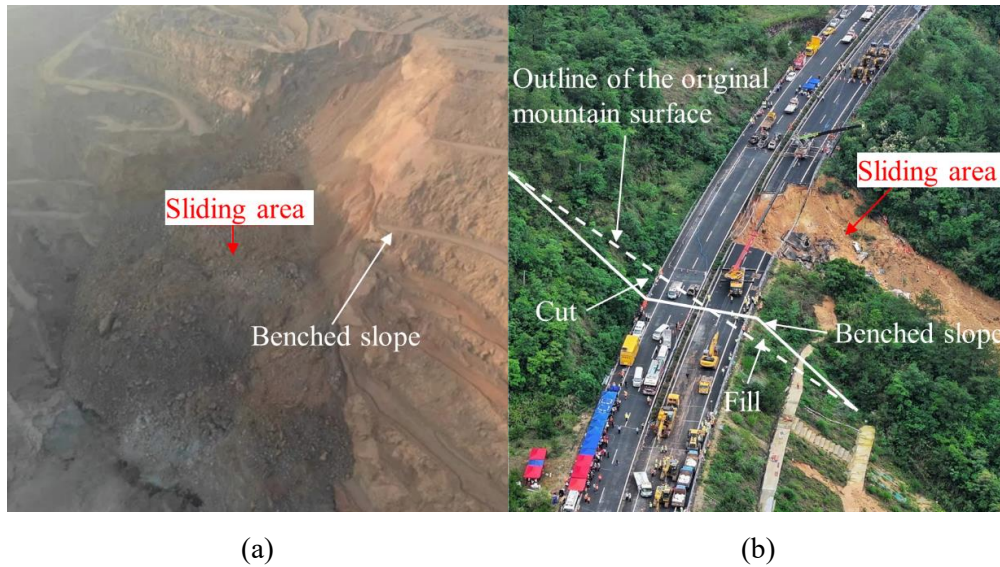


Fig. 1-2 Landslides related to benched slopes: (a) collapse accident at the open-pit coal mine of Inner Mongolia Province, China (ScienceView, 2023); (b) Mei–Da Expressway in Guangdong Province, China (Yue, 2024);

Given the widespread use and critical importance of benched slopes in hilly real estate developments, highway construction, and the mining industry, ensuring their stability is essential. Therefore, it is crucial to examine the relationship between benched slope instability and its geometric configuration. Additionally, further studies are needed to examine the variations in the stress and strain fields within a benched slope, as well as the mechanisms of failure under different loading conditions.

1.2 Motivation

This research was motivated by an engineering project at the Mae Moh mine, which is the largest surface coal mine of Thailand. The mine supplies 10–15 million tons of lignite annually to five coal-fired power plants operated by the Electricity Generating Authority of Thailand (EGAT) in Mae Moh, Lampang, in northern Thailand. These power plants produce approximately 20% of Thailand’s total electricity (Mavong et al., 2013), thus playing a significant role in the country’s energy transition process.

An excavation project is underway at the Mae Moh mine, employing benching techniques for the pit wall. The final pit wall inclination and batter slopes were specifically designed according to the strength

characteristics of each rock layer. While the bench height was uniformly maintained at 11 m, the bench width was adjusted to achieve an overall slope angle of 18° (corresponding to a vertical-to-horizontal ratio of 1:3). The batter slope was set at 45° for gray claystone and 14° for the topsoil or red bed layer.

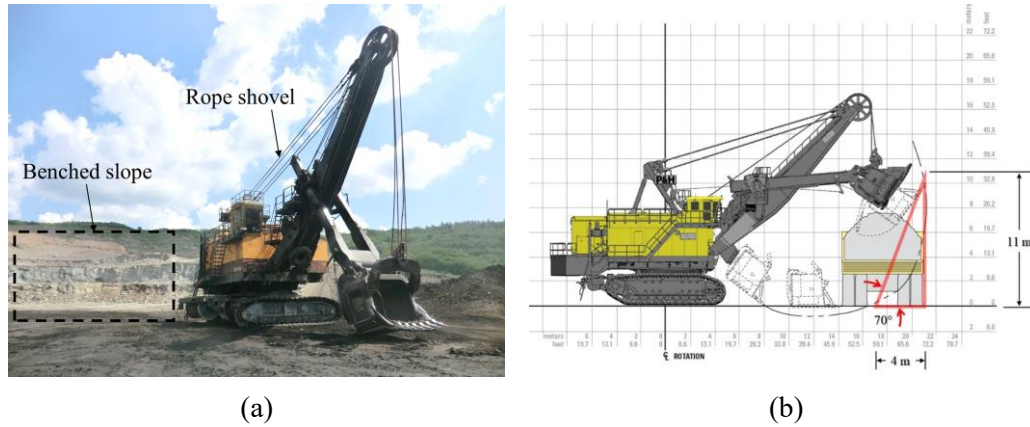


Fig. 1-3 Batter slope formed by shovel's excavation: (a) electric rope shovel (b) diagram of the operation process of the shovel (KomatsuMiningCorp., 2020)

Before reaching the final pit wall, the batter slope was initially set at a steeper angle than in the final pit design. According to the geotechnical report for the Mae Moh mine (EGAT, 1985), the recommended batter slope is 70° for bench heights up to 10 m, and 45° for bench heights up to 20 m. Typically, electric rope shovels created batter slopes of approximately 70° , with a cut height of up to 19 m and a radius of 24 m in gray claystone. As noted, the bench height was consistently maintained at 11 m (see **Fig. 1-3**).

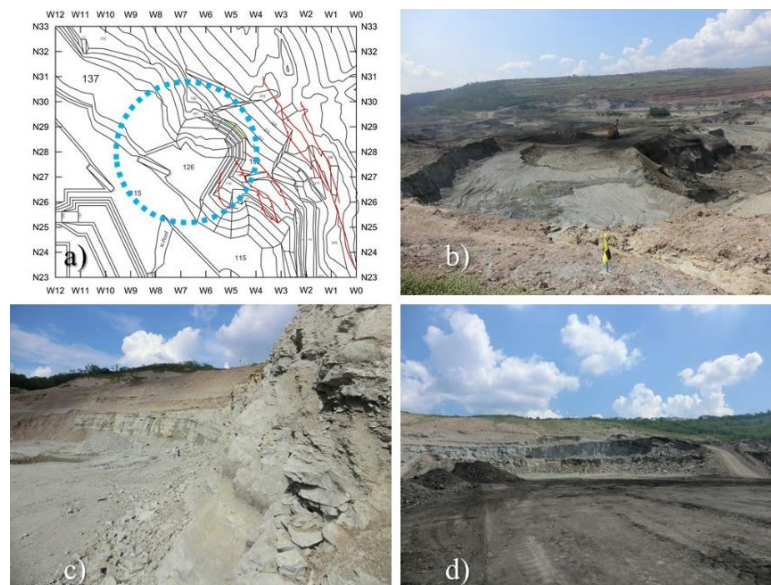


Fig. 1-4 Mine plan and working face photographs of sub-mining area Area 4.1: (a) modified mining plan (courtesy of Leelasukseree et al. (2021)); (b) overview of Area 4.1, looking South; (c) close-up view of the working face; (d) overview of Area 4.1, looking Northeast

Fig. 1-4 presents a portion of the mining plan, including photographs of a sub-mining area, designated as Area 4.1, taken between 2011 and 2017. The working face comprised gray claystone interburden with a batter slope of 70° , marked by a blue dashed circle. According to site engineers' observations, the 70° batter slope remained stable during mining operations, likely for a period not exceeding a few years.

A stability analysis of the working face, particularly the temporary gray claystone slope with a 70° batter angle, is necessary. Due to the complicated outline of the benched slope and the influenced stress and strain by interactions among different benches, evaluating its stability is challenging. The factor of safety (*FS*) of benched slopes altered by its failure mode, which can be varied by bench configurations and material properties, ranging from sliding of the entire slope (see **Fig. 1-2 (a)**) to localized sliding on one or several benches (see **Fig. 1-2 (b)**). Some researchers have attempted to understand the failure mechanisms of benched slopes using limit equilibrium methods based on global failure modes (Kaniraj & Abdullah, 1993) or by assuming that all failure surfaces pass through the slope toe (Pantelidis & Psaltou, 2013). However, these approaches may be overly simplified, as they do not fully account for the potential failure patterns of benched slopes under different conditions. Consequently, these analytical models may not accurately capture the true behavior of benched slopes due to a lack of experimental support.

Furthermore, many previous studies on benched slopes have been based on two-dimensional (2D) assumptions, disregarding boundary effects, which can lead to an underestimation of stability by overlooking lateral resistance. Additionally, observations of benched slope failures are often limited to a macroscopic level, leaving the underlying particle-level mechanisms insufficiently explored.

Based on the background outlined above, the theoretical and experimental foundation of current evaluation methods for benched slope safety is insufficient. Consequently, benched slopes in engineering projects are often designed empirically, with excessive redundancy. This approach is not cost-effective and may lead to severe outcomes. Therefore, this research aims to comprehensively study the instability of benched slopes, considering critical influencing factors.

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Chapter 2 Literature review and methodology

This chapter comprehensively reviews the existing literature on benched slope stability evaluation to define the scope of this study and introduces the methodology to be employed.

In **Section 2.1.1**, Culmann's method is introduced, including its definition and applications, as a foundation for the theoretical derivations in the subsequent chapters.

In **Section 2.1.2**, theoretical considerations on benched slope stability are presented, focusing primarily on two main approaches: the Limit Equilibrium Method (LEM) and the Limit Analysis Method (LAM). Both methods have evolved from two-dimensional (2D) to three-dimensional (3D) conditions. Additionally, assumptions regarding failure modes and the effects of various external excitations influence the development of these theoretical frameworks.

In addition to analytical methods, numerical simulations on this topic are reviewed in **Section 2.1.3**. Both 2D and 3D approaches using the Finite Element Method (FEM), Finite Difference Method (FDM), and Distinct Element Method (DEM) are employed to gain meaningful understanding and facilitate comparisons with theoretical predictions.

Experiments on benched slopes are summarized in **Section 2.1.4**, including centrifugal model tests and 1G shaking table tests combined with wave analysis and image processing techniques. While experimental validation of theoretical and numerical results is crucial, there remains a scarcity of experiments focused on benched slope.

Section 2.2 introduces the fundamental mechanisms of the primary methodology employed in this study.

From the information above, **Section 2.3** identifies the research gap for benched slope stability, outlines the research objectives, and presents the thesis structure.

2.1 Literature review

2.1.1 Evaluation of steep slope stability by Culmann's method

Golder (1948) noted that, building on Coulomb (1773) work on the critical height of a vertical cut in cohesive soil (see **Eq. (2-1)**), Francais (1820) and Culmann (1866) independently developed similar methods to calculate the maximum height of a finite uniform slope with a given angle (see **Eq. (2-2)**), based on the assumption of planar slip or translational failure.

$$h = \frac{4c}{w} \frac{1 + \sin\phi}{\cos\phi} \quad (2-1)$$

$$\beta = \sin^{-1} \left(\frac{Hw(1 - \cos(\beta - \phi))}{4c \cos\phi} \right) \quad (2-2)$$

Where,

h : critical height of a vertical cut of cohesive soil

c : cohesion

ϕ : internal friction angle

w : unit weight

β : critical slope

H : slope height

Taylor (1948) observed that this approach is suitable for steep slopes but does not provide satisfactory predictions for flatter slopes. This finding was later confirmed by Salmasi et al. (2019), who compared the factor of safety (FS) calculated using Culmann's method with values obtained from slice methods involving circular failure surfaces (see **Fig. 2-1**). They found that Culmann's method consistently produces higher FS values; however, the discrepancy decreases as the slope inclination increases. Additionally, Salmasi et al. noted that Culmann's assumed failure surface, which passes through the slope toe, aligns with simulation results for steep slopes.

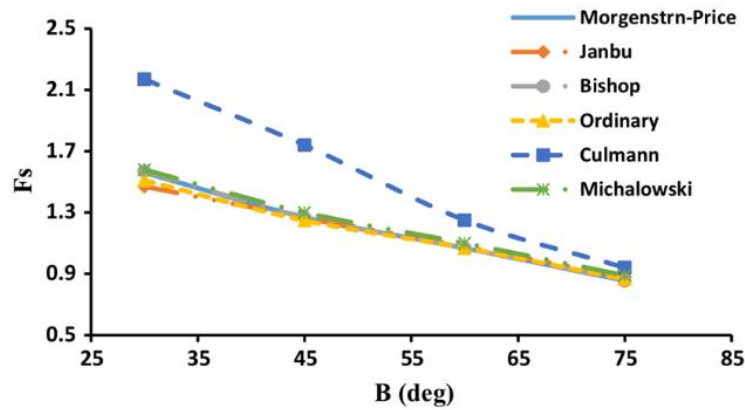


Fig. 2-1 Comparison of six methods for FS for slope under dry conditions with $\phi = 14^\circ$, $c = 15 \text{ kN/m}^3$, $H = 8 \text{ m}$ (Salmasi et al., 2019)

In applying Culmann's method to steep cemented sand cliffs, Hampton (2002) identified discrepancies between theoretical predictions and observed site conditions regarding cliff height and slope inclination. Collins (2004) further noted that the method overestimated the extent of crest retreat and underestimated the angle of the failure plane. To address these limitations, Collins and Sitar (2011) developed a simplified analysis based on infinite slope assumptions, incorporating a prescribed failure plane parallel to the slope inclination and accounting for toe height and crest tension cracks.

Despite the limitations of Culmann's method, it remains useful for preliminary design, particularly for steep slopes. Additionally, due to its simplified planar failure surface assumption, this type of wedge stability analysis is widely applied to pseudo-static slope stability problems. Some researchers have extended this approach to scenarios with prescribed sliding planes. For instance, Ling and Cheng (1997) studied rock sliding along a joint plane with tension cracks and pore pressure, under seismic or blasting excitation. Cecconi et al. (2017) considered an analogous scenario with additional external force applied by anchors.

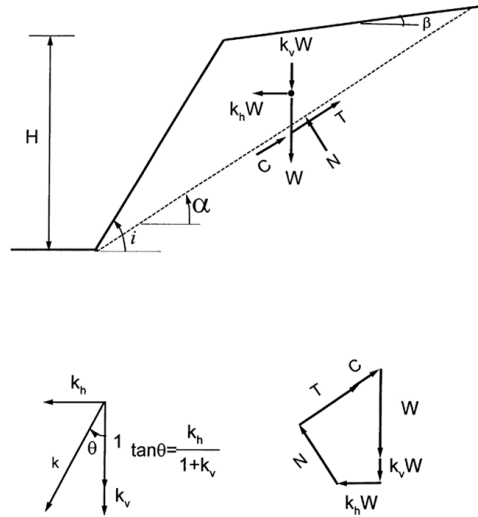


Fig. 2-2 Seismic stability analysis with planar failure surface (Ling et al., 1999)

Without a preselected failure surface, Ling et al. (1999) extended Culmann's method to dynamic conditions in a homogeneous slope by incorporating both horizontal and vertical accelerations, as well as an inclined backfill (see **Fig. 2-2**). They derived a closed-form theory for the critical inclination of failure plane, α_{cr} , and the stability number, N , as shown in **Eqs. (2-3), (2-4)** and **(2-5)**.

Where,

i : slope inclination

ϕ : internal friction angle

θ : angle between the resultant acceleration and vertical direction

k_h : seismic coefficient in the horizontal direction

k_v : seismic coefficient in the vertical direction

β : backfill inclination

$$\alpha_{cr} = \frac{i + \phi - \theta}{2} \quad (2-3)$$

$$\tan \theta = \frac{k_h}{1 + k_v} \quad (2-4)$$

$$N = \frac{1 + k_v \cos \beta (1 - \cos(\phi - i - \theta))}{4 \cos \theta \cos \phi \sin(i - \beta)} \quad (2-5)$$

Choudhury and Modi (2008) divided a slope into several finite horizontal slices to examine the reinforcement impacts on seismic instability analysis. Their results aligned closely with those obtained by other methods employing non-planar failure surfaces, even for flat slopes under seismic conditions. Christian and Urzúa (2017) explored the relationship between pseudo-static and static FS in cases involving sliding along a slip surface. Given the static FS , the critical seismic coefficient in the horizontal direction, $k_{h,crit}$, can be obtained, as shown in Eq. (2-6), where α represents the failure surface angle. However, in the static Culmann method, FS can only be determined through iteration, rather than an explicit expression, due to the unknown failure surface angle when FS is not equal to one.

$$k_{h,crit} = \frac{FS - 1}{\tan \phi + 1 / \tan \alpha} \quad (2-6)$$

2.1.2 Theoretical analysis of benched slopes

2.1.2.1 Two dimensional (2D) Limit Equilibrium Methods (LEM)

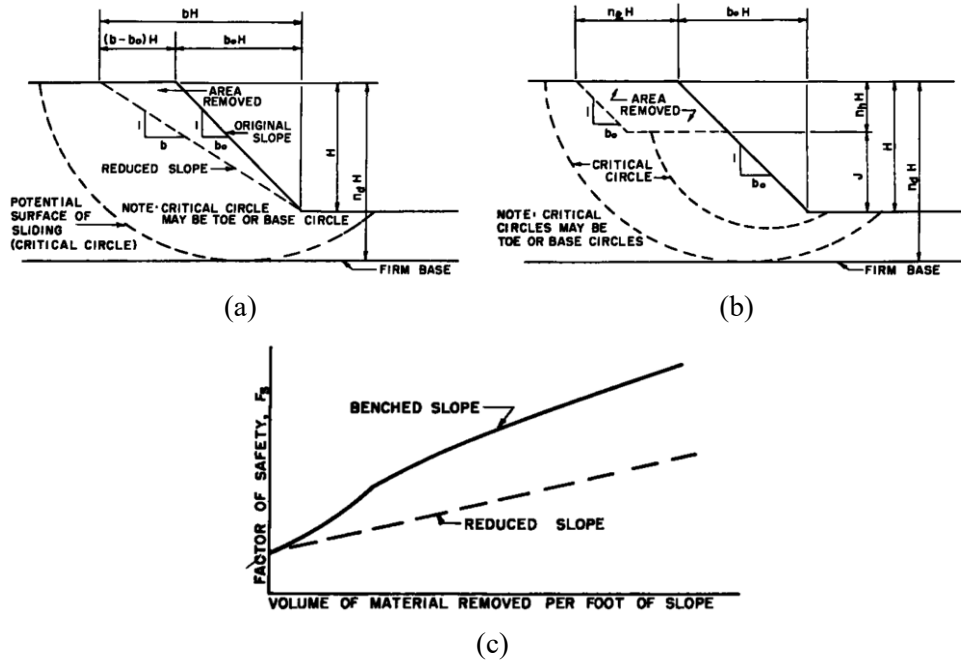


Fig. 2-3 Comparison between the slope reduction and the benching: (a) slope reduction; (b) benching; (c) FS increases with different earth removal methods (Hennes et al., 1959)

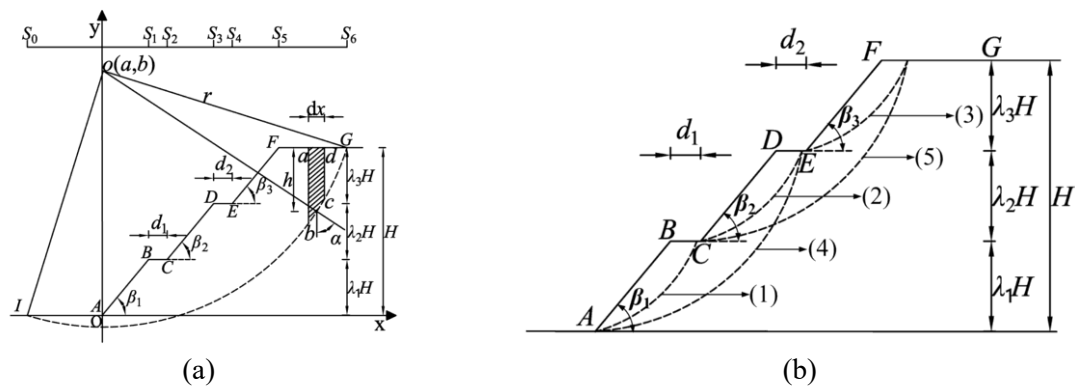
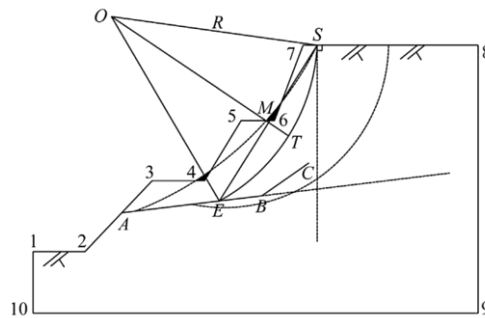
Research on benched slope stability dates back several decades. To guide remedial excavation on slopes, Hennes et al. (1959) compared the effects of slope reduction and benching using the Swedish method with toe or base circles (see **Fig. 2-3 (a)** and **(b)**). They observed that FS increased with both methods at different rates, as illustrated in **Fig. 2-3 (c)**. Consequently, they concluded that benching was more effective per unit of excavation.

Fig. 2-4 Embankment on soft soil with no tension crack

For an embankment with a berm (see **Fig. 2-4**), which can be considered a type of benched slope, Kaniraj and Abdullah (1993) derived closed-form solutions to determine the location of the critical sliding circle and the minimum FS , based on a specified limiting tangent and moment equilibrium. In their derivation, the berm segment ABCE provided additional resisting moment, thereby enhancing the stability of the embankment.

Fig. 2-5 Example: Geometry of earth slope with three benches (Pantelidis & Psaltou, 2013)

benched slopes and presented stability charts accommodating a wider range of bench geometries.



Bekele and Meten (2022) observed that the global FS increased with the number of benches using the general limit equilibrium method in Slide 6 software. Yang and Zhu (2023) argued that traditional circle center grid search methods, which rely on an arc or logarithmic spiral in most LEM methods, lack specific physical meaning. Additionally, searching for the rotation center's position is time intensive. In order to address these issues, they developed a sliding surface search model based on the brachistochrone principle, which requires only the locations of the endpoints of the sliding curve. Their work employed a simplified Janbu method.

parameters, forming a database to train a radial basis function neural network (RBFNN) model, enabling predictions of controlling slope stability. To address stability issues in benched coal mine waste rock dumps, where high heterogeneity in shear strength parameters presents challenges, Kumar et al. (2023) integrated probabilistic methods—namely the Monte Carlo Method (MCM) and the Alternative Point Estimate Method (APEM)—into the General Limit Equilibrium (GLE) method to analyze the probability of failure

2.1.2.2 Two dimensional (2D) Limit Analysis Methods (LAM)

In addition to the Limit Equilibrium Method (LEM), the Limit Analysis Method (LAM) has also been employed for the stability assessment of benched slopes. One approach within LAM is the Upper Bound Method, which estimates the collapse load by assuming a potential failure mechanism or mode. This method calculates the external work done by the loads within this failure mode and balances it against the internal energy dissipation, thereby providing an upper limit for the maximum load.

Chen (1975) is credited as the first to apply this technique to a problem similar to that of benched slopes—a two-stage, non-homogeneous anisotropic slope with a logarithmic spiral failure surface, where the earth removal action on the crest influenced calculations in both the external work rate and the internal energy dissipation rate (see **Fig. 2-8**).

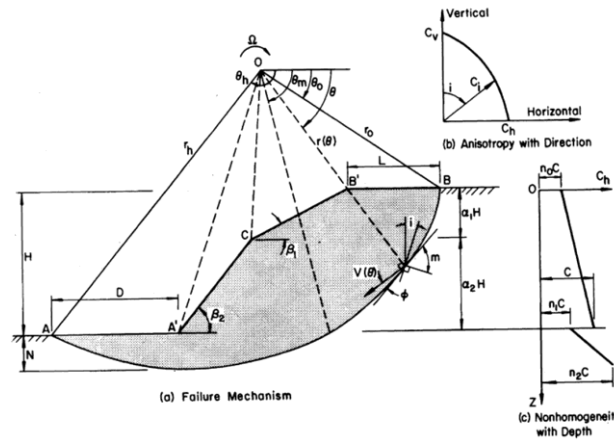


Fig. 2-8 Failure mechanism of the two-stage slope with log spiral sliding surface (Chen, 1975)

Gao et al. (2014) derived the FS for a slope with multiple benches using the Upper-Bound Method in combination with the Shear Strength Reduction (SSR) method. In the theory, the potential slip surface was identified by sequential quadratic programming, and findings indicated that local instability became more pronounced as the slope angle increased. Based on this approach, Zhang and Cheng (2017) extended stability charts from the stability tables developed by Pantelidis and Psaltou (2013), using the graphical method of Klar et al. (2011) for homogeneous slopes with benches, based on the Upper-Bound Method and SSR. These charts enable faster and more accurate stability assessments of benched slopes. Li and Xiao (2022) evaluated the seismic stability of a two-stage soil slope reinforced with stabilizing

piles (see **Fig. 2-9**) and derived an explicit expression for the required pile shear resistance.

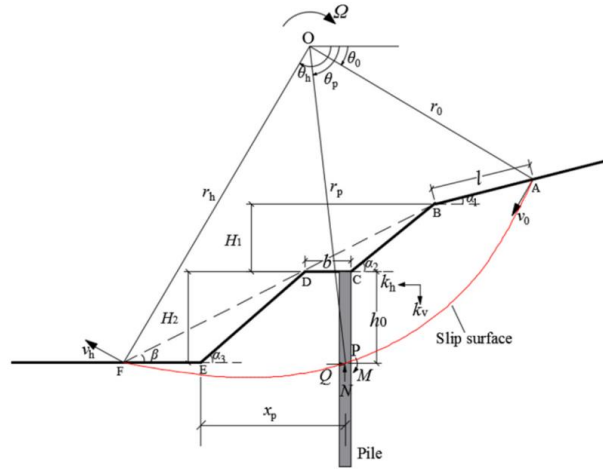


Fig. 2-9 Analysis model of rotational failure mechanism of a two-stage slope reinforced with one row of piles under seismic conditions (Li & Xiao, 2022)

2.1.2.3 Three dimensional (3D) Limit Equilibrium Methods (LEM)

The analytical methods mentioned above were conducted under 2D conditions. These models are commonly used because they are simpler to construct and provide faster computation times compared to 3D models. However, they tend to produce conservative results.

3D stability analysis for benched slopes has also been developed. Using the 3D LEM software Slide3, Bar Neil et al. (2020) conducted failure evaluations for open-pit slopes after creating 3D models via unmanned aerial vehicle (UAV) photogrammetry (see **Fig. 2-10**). They noted that this approach facilitates better integration with mine planning, as most of coal mines had previously been planned through kinematic or 2D LEM analyses.

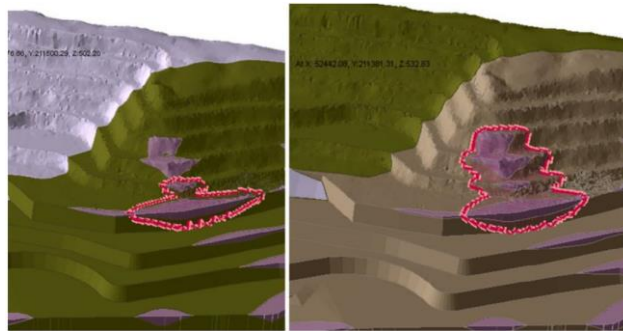


Fig. 2-10 3D LE models evaluating stability of future pit slope design (Bar Neil et al., 2020)

2.1.2.4 Three dimensional (3D) Limit Analysis Methods (LAM)

In the past decade, several 3D methods for benched slope stability have been developed based on the LAM. Yang and Long (2016) applied the horn-shaped 3D failure mechanism proposed by Michalowski and Drescher (2009), thereby extending the analysis of a two-stage slope into 3D conditions (see **Fig.**

2-11). Their study also incorporated seismic inertia forces, enhancing LAM's capability to estimate dynamic slope stability.

Additionally, Yang and Xu (2017) accounted for strength inhomogeneity in their analysis of a two-stage slope. Xu et al. (2018) examined the combined effects of nonlinearity and dilatancy, finding that both the depth coefficient and the dilative parameter positively influence slope stability. Yang et al. (2021) investigated a benched slope reinforced with piles, while L. Wang et al. (2024) studied a similar scenario, considering the effect of suction in unsaturated soil.

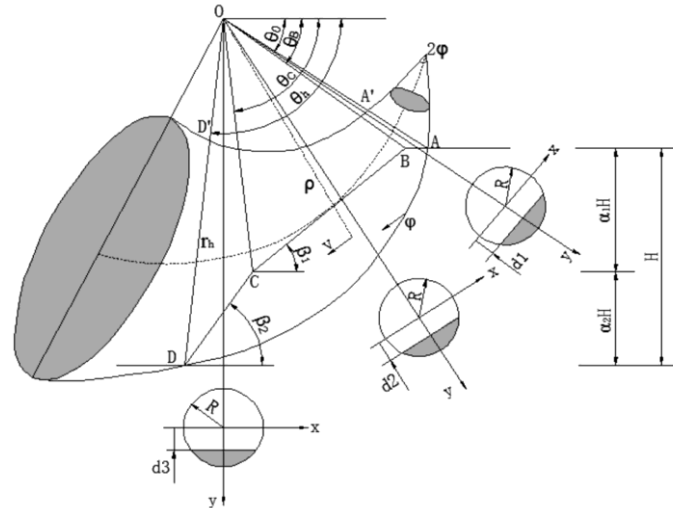


Fig. 2-11 Horn-shape surface of 3D rotational mechanism (Yang & Long, 2016)

2.1.3 Numerical simulation of benched slopes

2.1.3.1 Two dimensional (2D) numerical simulation

With advancements in computing efficiency, various numerical methods are becoming increasingly popular in slope stability engineering. The Finite Element Method (FEM), Finite Difference Method (FDM), and Distinct Element Method (DEM) are frequently employed for this purpose and can be combined with the Shear Strength Reduction (SSR) to acquire the *FS* for benched slopes.

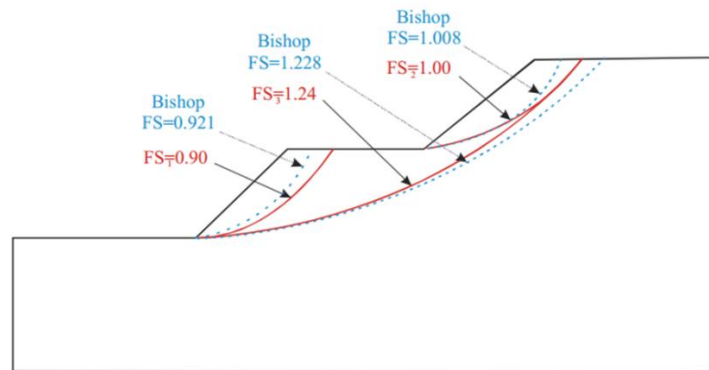


Fig. 2-12 Several slip surfaces identified in benched slope byMSSR and LEM (Cala & Flisiak, 2003)

To evaluate the *FS* for various potential failure surfaces of a benched slope and to compare the results

with LEM, a modified shear strength reduction technique (MSSR) was implemented in the Finite Difference Method software (FLAC) after identifying the initial slip surface. This approach produced results closely aligned with those obtained using Bishop's method (Cala & Flisiak, 2003; Cala et al., 2004), as shown in **Fig. 2-12**.

Maknoon and Aubertin (2020) applied a coupled seepage–stress distribution–slope stability procedure to assess the stability of waste rock piles. The first two steps generated initial stress and pore water pressure distributions. It enables the calculation of effective stress in the third step. Finally, the *FS* was evaluated using FEM software. Their findings indicated that the external geometry of the pile had a greater impact on stability than rainfall, suggesting that uniform slopes should be replaced with benched slopes for improved stability.

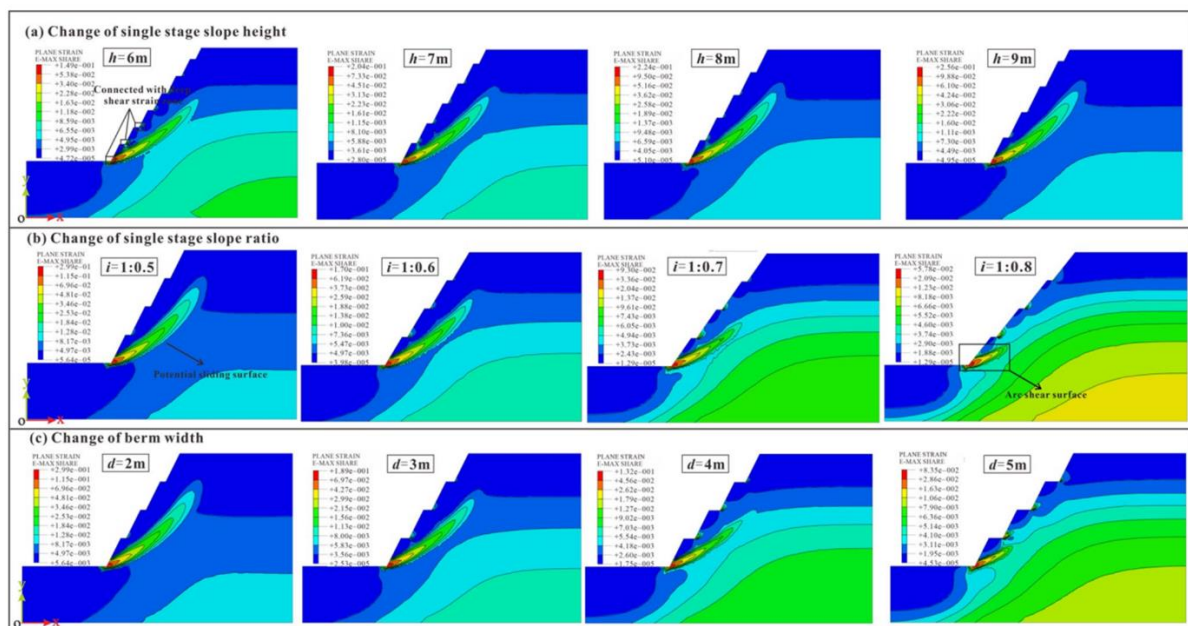


Fig. 2-13 Shear strain distribution of multi-stage slope with different slope shape parameters (Guo et al., 2023)

Singh et al. (2021) used FLAC 2D to compare the stability of railway slopes before and after reinforcement with soil nails. Mekonnen (2021) also employed FEM-based Plaxis software to study geometric modifications of road cut slopes through benching. Cao et al. (2023) utilized FLAC/Slope software in combination with the GPCC program and the WEPP model to balance slope-cutting and vegetation-planting strategies for high and steep cut slopes in the loess hilly region under future climate scenarios. For multi-stage loess slopes, Guo et al. (2023) analyzed the impact of slope shape parameters using FEM. They found that increasing single bench height led to higher shear strain of the slope toe and decreases shear failure in the upper section (see **Fig. 2-13 (a)**). The maximum shear strain arc extended with increasing slope inclination and decreasing bench width, as shown in **Fig. 2-13 (b)** and **(c)**. Expanding the bench width allowed the bench toe to share more shear force, effectively isolating

each bench from the others. Setting a wider platform at half or two-thirds of the slope height reduced concentration of stress occurring at the toe and limited global slope failure (see **Fig. 2-14**). Utilizing FEM software Phase 2.0, Garo et al. (2024) recommended combining slope reduction and benching as an effective slope stabilization strategy.

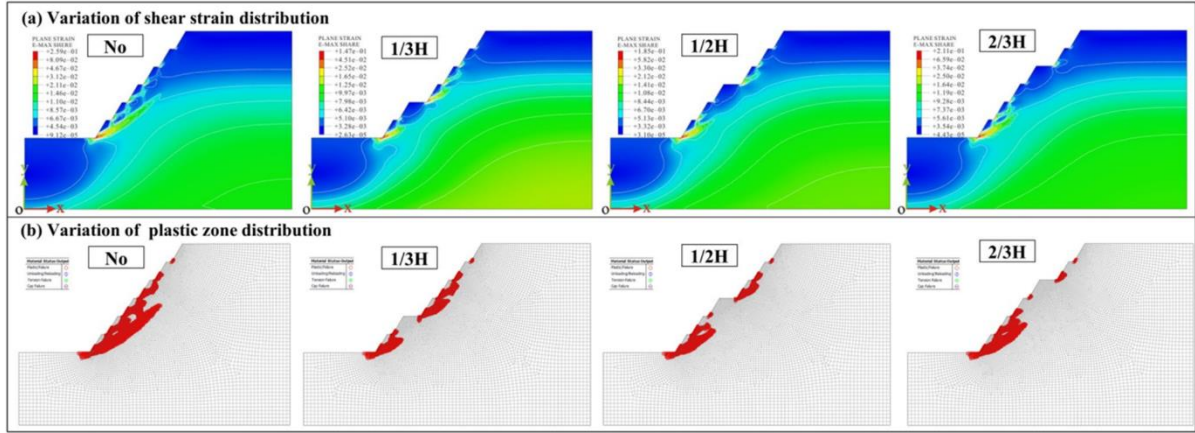


Fig. 2-14 Shear strain and plastic zone distribution of the slope with different platform positions (Guo et al., 2023)

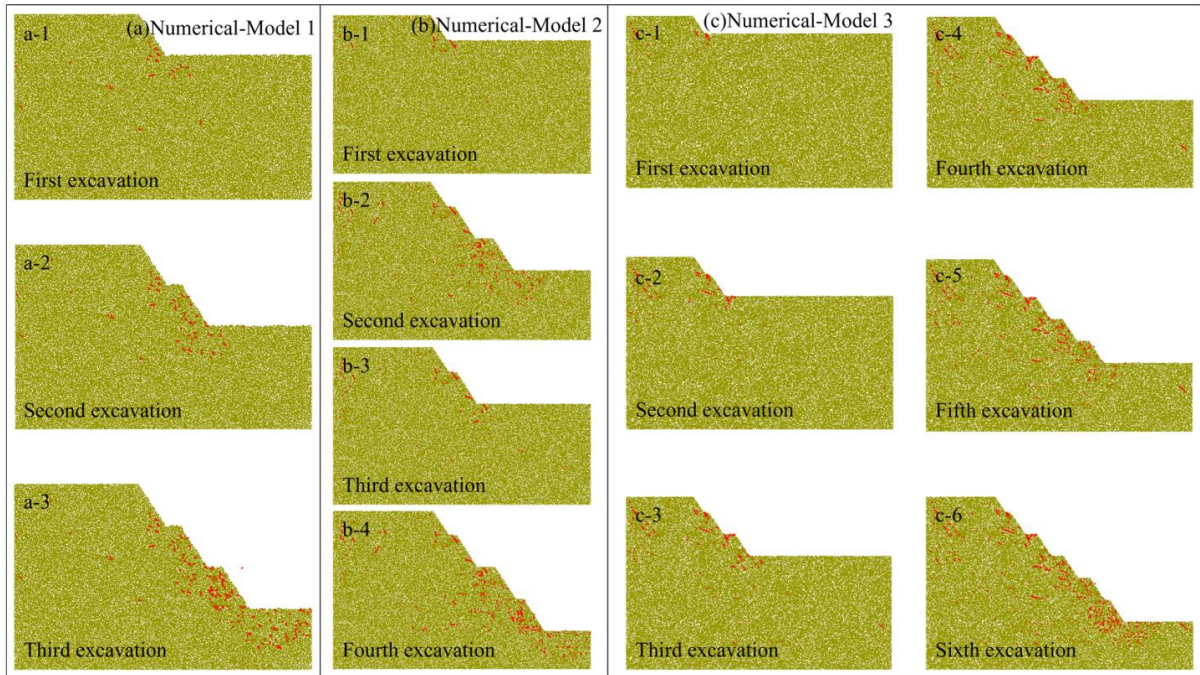


Fig. 2-15 Spatial and temporal evolution of cracks in the process of slope excavation (J.-m. Wang et al., 2024)

A DEM simulation of benched slopes constructed through excavation was performed by J.-m. Wang et al. (2024) to compare with results from the base friction test method. The evolution of cracks in three

DEM models throughout the excavation process is shown in **Fig. 2-15**. Most cracks were due to tensile stresses and propagated from the toe to the crest. Additionally, Zhao et al. (2024) investigated the impact of vehicle loads on benched slope stability.

2.1.3.2 Three dimensional (3D) numerical simulation

Javankhoshdel et al. (2021) used 3D FEM analysis software RS3 to analyze an open-pit mine, comparing the mode of failure and *FS* with results obtained from 3D LEM analysis in Slide3 software. The results from both methods showed good agreement (see **Fig. 2-16**).

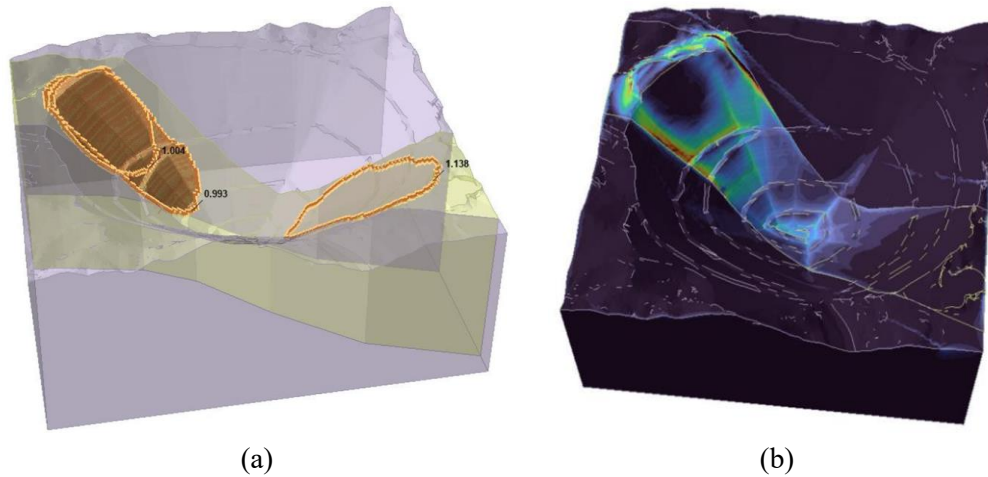


Fig. 2-16 Comparison between the 3D LEM and 3D FEM for an open pit mine: (a) results of the LEM analysis using MMO approach; (b) maximum shear strain contour for the FEM analysis (Javankhoshdel et al., 2021)

Li and Xiao (2022) utilized the FLAC3D program to verify their proposed calculation method for determining the required shear resistance of piles under varying seismic conditions. Under seismic excitation, the sliding surface moved inward within the slope, and the length of the pile below the slip surface decreased, as shown in **Fig. 2-17**.

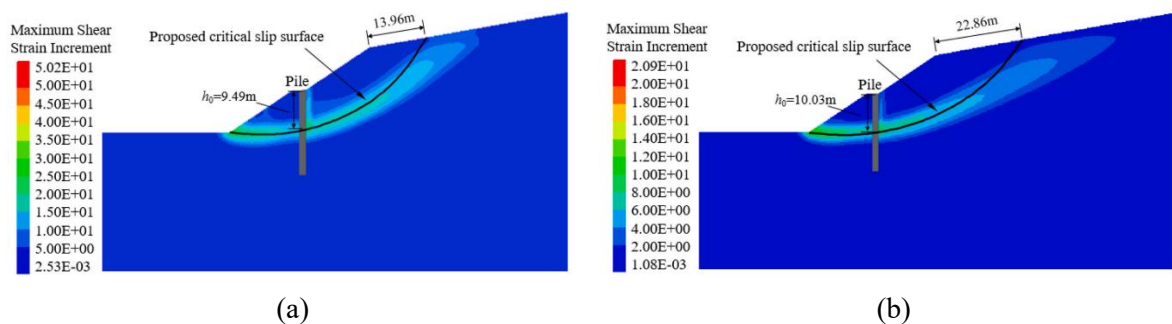


Fig. 2-17 The critical slip surfaces: (a) under static state; (b) under the seismic condition with $k_h=0.1$ and $k_v=0.05$ (Li & Xiao, 2022)

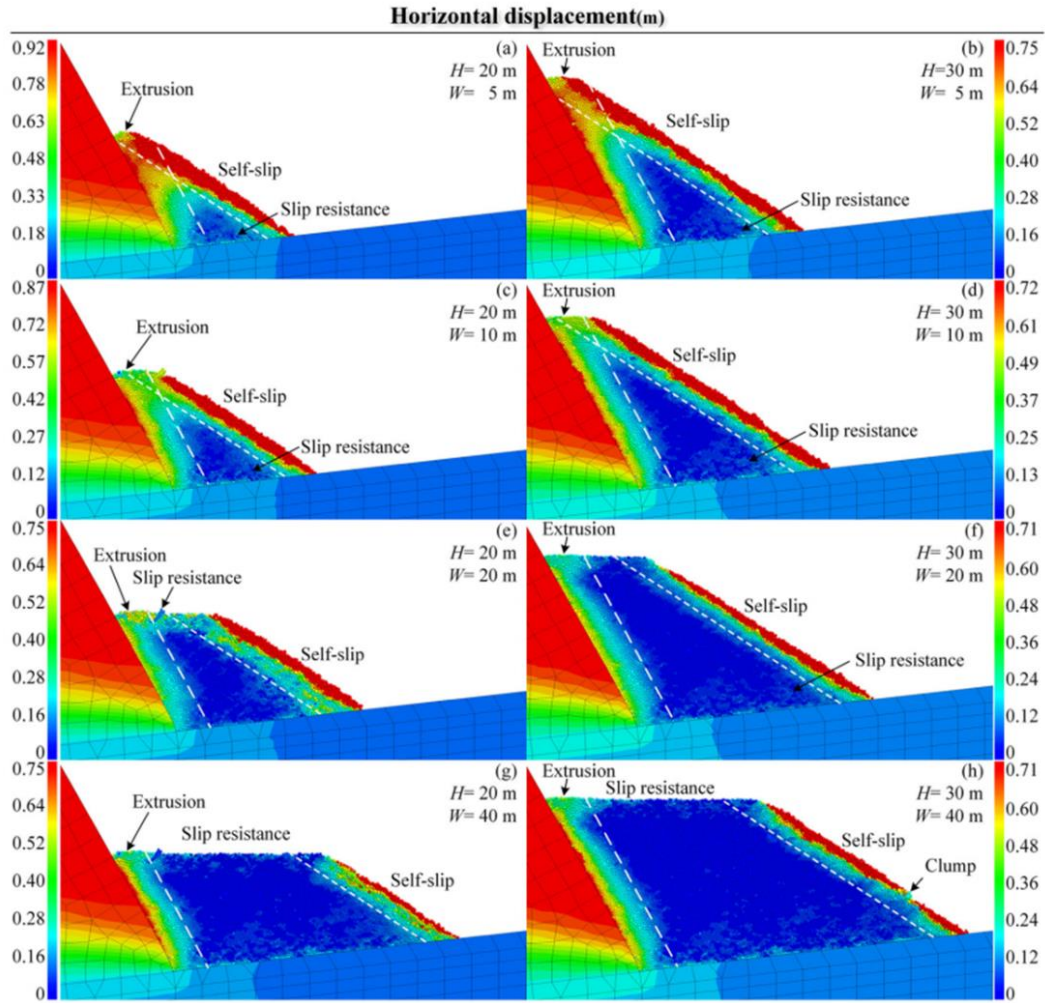


Fig. 2-18 Deformation characteristics of inner dump bench (Tian et al., 2022)

Tian et al. (2022) applied a discrete–continuous coupling method using PFC and FLAC3D to investigate the control mechanisms of end-slope deformation in open-pit mines. They found that increasing the width and height of the dump bench at the toe restrained end-slope deformation. Additionally, they classified the dump into three zones, based on displacement characteristics, that contribute to resisting slip forces from the end-slope: the “Extrusion Zone,” “Slip Resistance Zone,” and “Self-Slip Zone” (see **Fig. 2-18**).

Taking into account material degradation caused by the freeze-thaw (F-T) process, Rezaei and Seyed Mousavi (2024) developed a model using FLAC3D. They found that as the weathering grade increased, the *FS* reduced exponentially in the highly weathered area, while in the moderately weathered part, it dropped linearly.

2.1.4 Experiments of benched slopes

Studies on the instability of benched slopes are primarily on theoretical derivation and numerical simulation, with limited experimental support. Notable exceptions include the work of Jie et al. (2018),

who demonstrated that decomposing a slope into secondary slopes with benches could enhance the utilization of soil cohesion, as verified through centrifuge testing and FEM numerical results. Luo et al. (2020) investigated the effect of geogrid reinforcement on a benched slope under differential settlement and superstructure loading using centrifugal modeling (see **Fig. 2-19**).

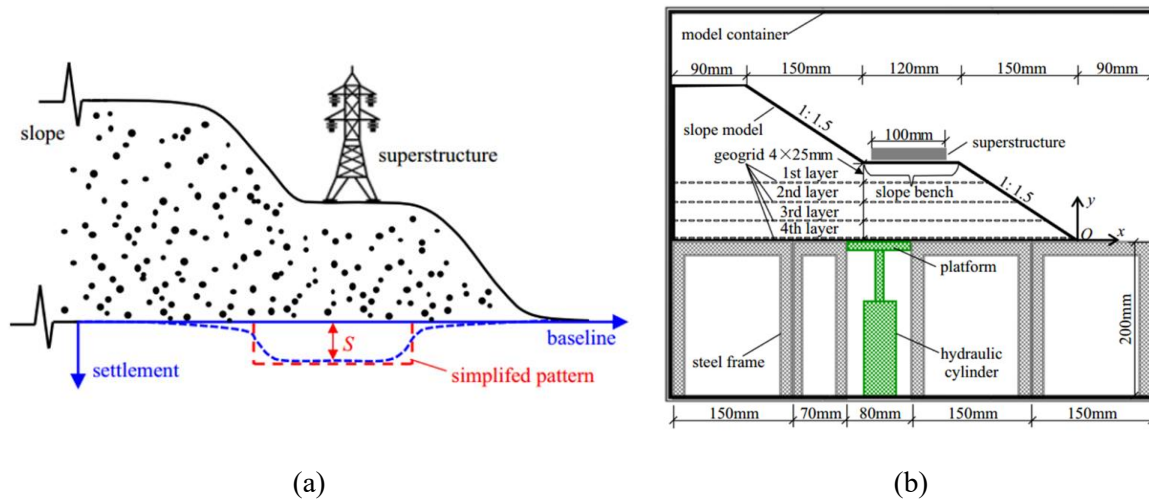


Fig. 2-19 Superstructures built on the benched slope subjected to differential settlement: (a) sketch of the test scenario; (b) schematic view of the centrifugal model (Luo et al., 2020)

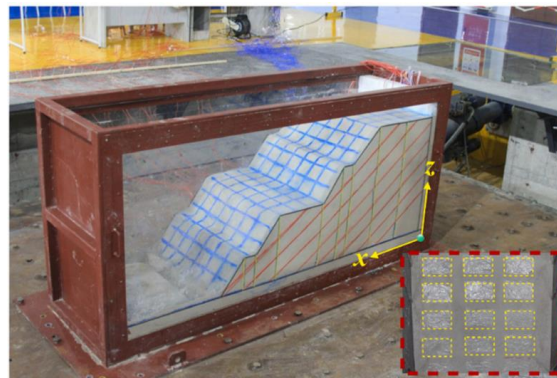


Fig. 2-20 The photo of the experimental model (Xin et al., 2024)

Shaking table tests at 1G were also conducted to evaluate the seismic stability of benched slopes. Xin et al. (2024) focused on dip and anti-dip rock slopes with structural planes (see **Fig. 2-20** and **Fig. 2-21**). They observed that the elevation amplification effect (EAE) was more pronounced in dip slopes than in anti-dip slopes (see **Fig. 2-22**); the dip slopes exhibited a higher energy release rate due to reduced damping and scattering of seismic waves. The accelerated failure was shown in dip slope, and the most obvious damage happened at the upper sections. In contrast, the anti-dip slope exhibited failure within the identical loading level, especially concentrated on the second bench. Lin et al. (2024) observed a similar phenomenon of seismic amplification in a three-stage slope with anchoring lattice beams and concluded that benching could reduce seismic response. In their experiment, the displacement pattern

was captured using Digital Image Correlation (DIC).

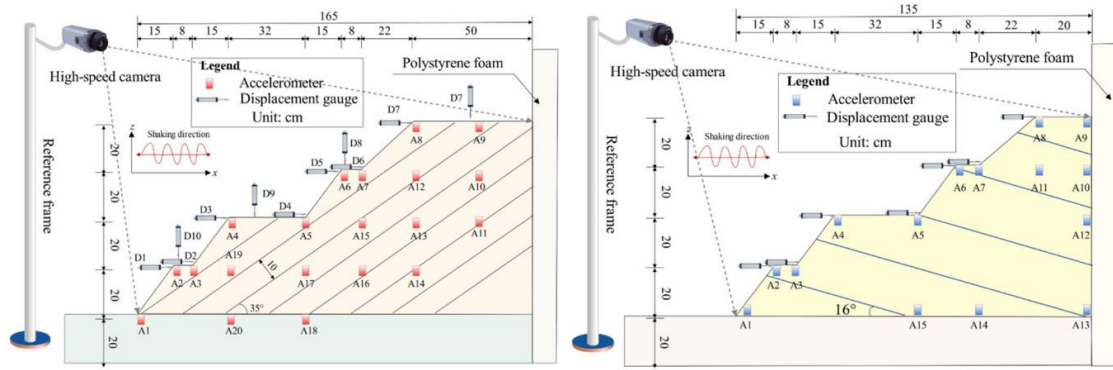


Fig. 2-21 Diagram of the dip and anti-dip slope models (Xin et al., 2024)

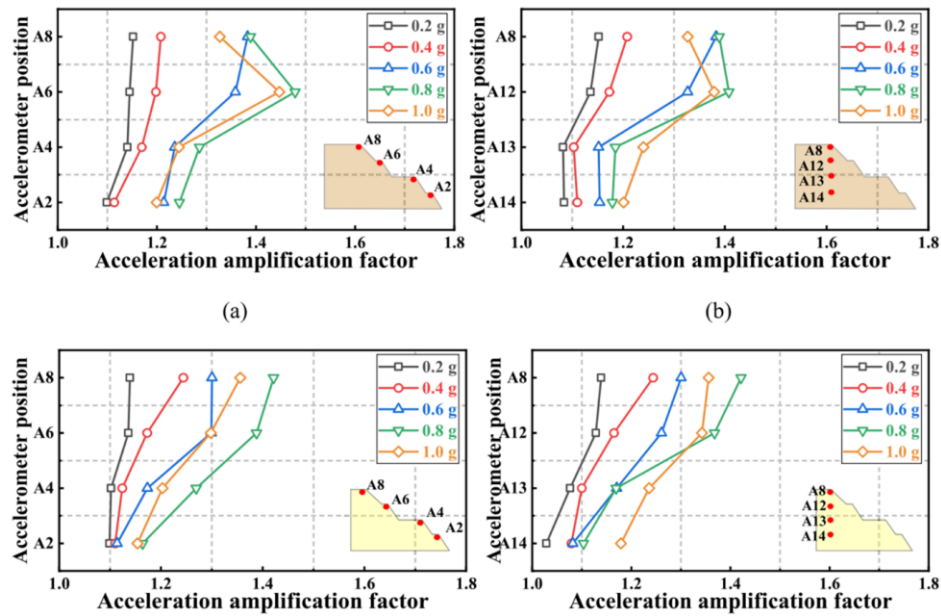


Fig. 2-22 Acceleration amplification factors of dip slope and anti-dip slope models: (a) AAFs of dip slope model for the slope surface; (b) AAFs of dip slope model for the inside of slope body; (c) AAFs of anti-dip slope model for the slope surface; (d) AAFs of anti-dip slope model for the inside of slope body (Xin et al., 2024)

2.2 Methodology

2.2.1 Geotechnical centrifuge

In geotechnical engineering, conducting destructive experiments at the prototype scale is typically uneconomical and hazardous. However, the stresses within earth structures and ground formations, such as slopes, are largely governed by the soil's own weight, affected by the dimensions. Furthermore, the soil's strength and deformation characteristics are strongly influenced by the magnitude of this internal stress, with strength increasing as internal stress rises. Consequently, when using a scaled-down model to predict slope behavior, the reduction in self-weight stress makes it hard to precisely project the

deformation behavior of the actual condition.

In the geotechnical centrifugal modeling, the model is scaled down to $1/n$ of the actual dimensions and applied with a centrifugal loading of n times normal gravitational acceleration, thereby reproducing the same stress conditions from self-weight as in the prototype. Here, n is referred to as the scaling factor. By applying centrifugal force, the reduced model experiences compressive and shear forces equivalent to those in the actual object, making it possible to similarly replicate the strength and deformation characteristics that counteract these stresses.

When conducting an experiment using a scaled-down model of the actual object, it is essential to consider the principles of similarity (see **Table 2-1**).

Table 2-1 Similarity laws of centrifuge test (ng is the centrifugal acceleration)

Parameter	Scaling factor (model/prototype)
Stress	1
Force	$1/n^2$
Length	$1/n$
Mass	$1/n^3$
Velocity	1
Acceleration (dynamic)	n
Frequency (dynamic)	n
Time (static)	$1/n$
Time (dynamic)	$1/n^2$

Since centrifuge model experiments are small-scale models conducted in a centrifugal field, they present issues common to general small-scale model experiments, along with challenges unique to centrifuge testing. These issues must be fully understood before conducting centrifuge model experiments (Igarashi, 2002).

- (1) Non-uniformity of centrifugal acceleration: the centrifugal acceleration is proportional to the distance from the center of rotation (see **Fig. 2-23**), so the model size must be sufficiently small compared to the centrifuge's rotation radius. However, in practice, creating a centrifuge large enough for this effect to be negligible is not feasible.
- (2) Interference of gravitational acceleration: there is always a gravitational acceleration perpendicular to the centrifugal acceleration, leading to the deviation of the resultant acceleration. **Fig. 2-24 (a)** shows the beam centrifuge in DPRI, Kyoto University. In **Fig. 2-24 (b)**, the resultant acceleration is inclined, forming an angle with the horizontal direction.
- (3) Local deformation: the width of the slide surface (shear band) that forms during ground failure is determined by the soil's physical properties and cannot be scaled down. Therefore, in a reduced model, local deformations are expected not to exhibit similarity.

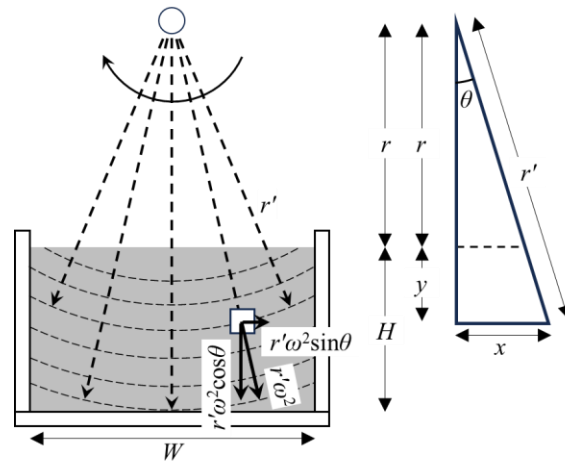


Fig. 2-23 The radial gravity field in a centrifugal model (modified from Sahare et al. (2020))

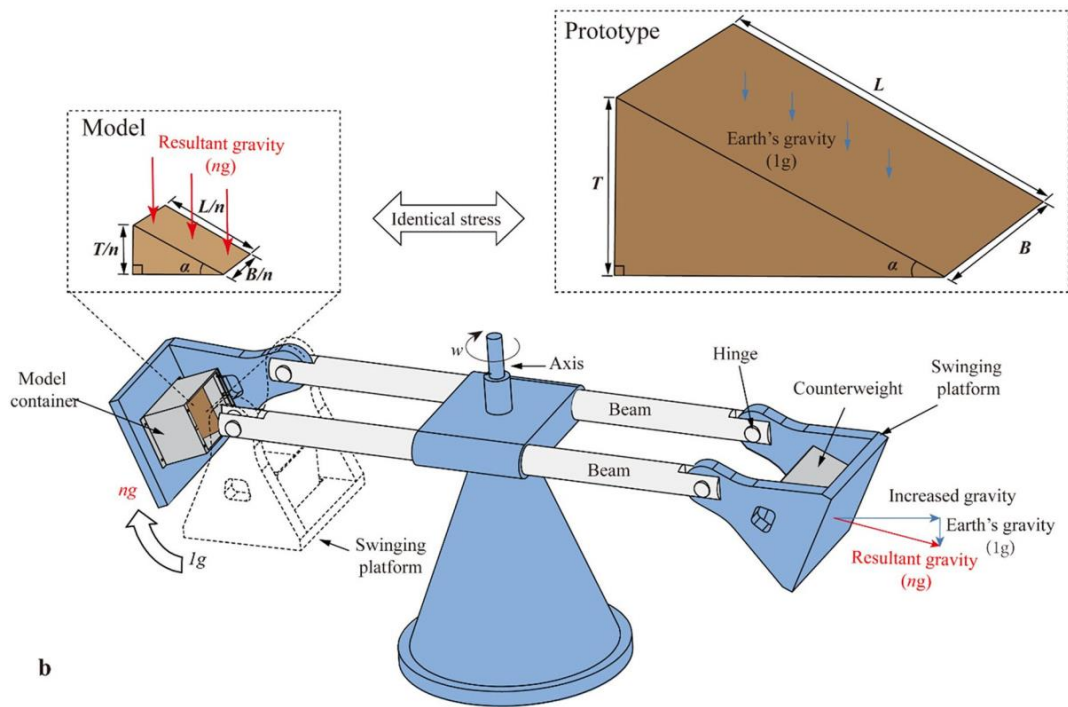
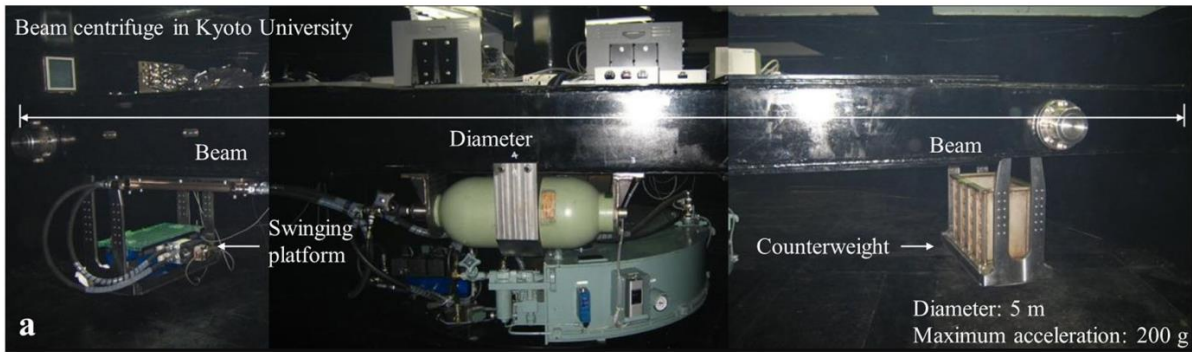


Fig. 2-24 Beam centrifuge: (a) beam centrifuge in Kyoto University from a plan view; (b) schematic view of the beam centrifuge (Fang et al., 2022)

2.2.2 Distinct element method (DEM)

Numerical simulation techniques, often adhering to the principles of continuum mechanics, are extensively employed. Nevertheless, the finite element method (FEM) faces significant limitations, including mesh distortion, particularly in scenarios involving large-scale deformation and structural failure. To model the large deformation process resulting from slope instability, various meshfree approaches, such as the material point method (MPM), have been developed. However, these methods, within continuum modeling theory, are constrained by phenomenological constitutive models that inadequately represent the microscopic mechanisms underlying slope failure. The discrete element method (DEM), which considers soil particles as fundamental units, has emerged as a vital tool for investigating and elucidating both macro and micro mechanisms of granular media. By inherently capturing contact and frictional interactions, DEM enables the simulation of the dynamic states of granular materials. Moreover, it can accurately replicate the actual grain size distribution while providing detailed particle-scale insights that are typically difficult to achieve through experimental methods (Wang et al., 2022).

The basic formulation for the distinct element method for granular materials was proposed and described by Cundall and Strack (1978) and Cundall and Strack (1979b) and a subsequent paper written by Cundall and Strack (1979a).

The calculation procedure in a DEM simulation is presented in **Fig. 2-25**. To initiate the simulation, the operator defines the geometrical information, sphere coordinates, boundary settings, and material properties. A loading schedule is determined, and the simulation runs as a dynamic analysis. During each time increment, particle contacts are recognized, and inter-particle forces are calculated based on contact distances. These forces allow for the calculation of resultant forces and moments, which in turn determine particle motion. Using central-difference integration, particle displacements and rotations are updated at each step, and this process continues with recalculated forces based on updated geometry (O'Sullivan, 2011).

DEM simulation has been applied on slope stability analysis (Cheng & Lau, 2014; Chiu & Weng, 2019; Hassan & El Shamy, 2019; Jin et al., 2024; Liu et al., 2022; Tao et al., 2022; Wang et al., 2022; Wang et al., 2020; Weng et al., 2015; Wu et al., 2020; Xie et al., 2024; Yang et al., 2023; Zhou et al., 2023). Nonetheless, when simulating centrifugal modeling by DEM, problems happened related to the scaling law. If the contact properties obtained by the unit test were applied to the simulation directly, unrealistic results could occur with much less deformation because they were not defined by stress or strain. Therefore, these parameters should be scaled and further revised by the modified scaling law (Weng et al., 2016; Weng et al., 2017). As presented in **Table 2-2**, deformation became larger with increasing modified parameter (a). In their study, its value took 0.3 through back analysis.

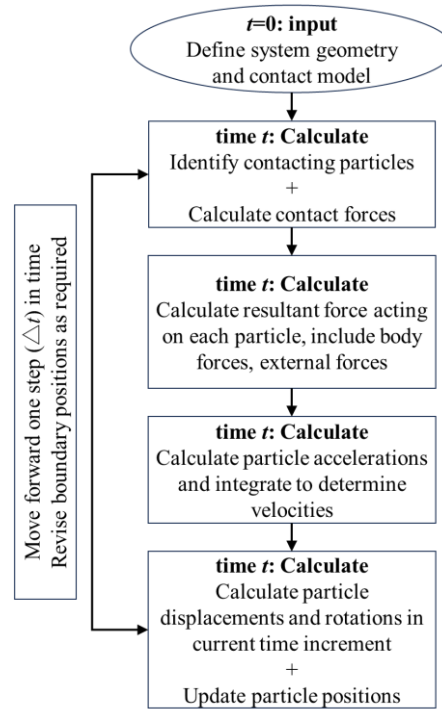


Fig. 2-25 Schematic diagram of sequence of calculations in a DEM simulation (modified from O'Sullivan (2011))

Table 2-2 Dimensional relationship of material properties of DEM analysis (Weng et al., 2017)

Physical Property	Unit	Scale factors (prototype/model)	Modified scale factors (prototype/model)
Linear dimension	m	N	N
Particle normal stiffness	kN/m	N	N^a
Particle shear stiffness	kN/m	N	N^a
Parallel-Bond normal stiffness	kPa/m	$1/N$	$1/N^a$
Parallel-Bond shear stiffness	kPa/m	$1/N$	$1/N^a$
Friction		1	1
Parallel-Bond normal strength	kPa	1	1
Parallel-Bond shear strength	kPa	1	1
Density	kg/m ³	1	1

2.3 Research scopes and thesis structure

The literature review in **Section 2.1** outlines the progression of previous studies on benched slopes, presenting fundamental research methods in this field and establishing a solid foundation for this study. However, certain limitations persist, which this thesis seeks to address using the methodology introduced in **Section 2.2**. The research gap, the research objectives, and the thesis structure are

presented as follows.

2.3.1 Research gap

- (1) **Section 2.1.1** reviews Culmann's method as applied to preliminary slope stability evaluation. This method is straightforward and reliable, particularly for steep slopes under seismic excitation. **However**, it conventionally uses the stability number (N) rather than the more intuitive factor of safety (FS), and calculating FS with previous methods often requires an iterative process. Thus, an explicit closed-form expression for FS is needed. Additionally, the traditional Culmann method is limited to two-dimensional (2D) conditions, which can result in conservative predictions. Furthermore, this method has not been applied to benched slopes, and its effectiveness should be validated for slopes with complex geometries.
- (2) **Section 2.1.2** reviews previous theoretical considerations on benched slope stability, introducing two main methods: the Limit Equilibrium Method and the Limit Analysis Method. These analytical methods can account for a broad range of influencing factors under both 2D and 3D conditions. **However**, most LAM studies focus solely on global failure modes due to the high computational demands of the Shear Strength Reduction (SSR) process. Similarly, LEM relies on complex searching algorithms to identify the critical sliding surface, which requires intensive computation and is challenging to apply in engineering practice. Additionally, the assumed failure modes have not been experimentally verified. Furthermore, neither method considers the effect of lateral resistance.
- (3) **Section 2.1.3** reviews various numerical methods applied to benched slopes, including the Finite Element Method, Finite Difference Method, and Distinct Element Method. Simulations using these methods provide detailed insights and enable comparisons with theoretical analyses. They are also valuable for parametric studies to identify optimal stability configurations for benched slopes. **However**, FEM and FDM cannot capture particle-level information. For DEM, only two-dimensional (2D) simulations have been applied to this topic, due to computational constraints. Therefore, conducting a three-dimensional (3D) DEM simulation is worthwhile.
- (4) **Section 2.1.4** reviews several experiments on benched slopes. Centrifuge modeling and 1G shaking table tests, combined with signal analysis and real-time image processing, provide valuable recording of the realistic reaction of benched slopes subjected to static and dynamic loadings. **However**, these experiments are limited in number and lack systematic coverage. Additionally, earth pressure inside the slope, a critical indicator of slope state variation, was not measured in these experiments.

2.3.2 Research objectives

- (1) The centrifugal tests will first be used to observe potential failure modes, critical height, and seismic resistance of benched slopes under various configurations and loading conditions.
- (2) The experimental results will then be validated using a newly proposed closed-form solution based

on Culmann's method, which incorporates boundary effects and pseudo-static forces. The stress-strain relationship recorded in the centrifugal modeling will provide insights into the evolution of slope conditions and influence the theoretical calculation of lateral resistance.

- (3) Simulated models based on 3D DEM will be developed to verify the experimental results and to identify additional influencing factors. These models can also reveal details not easily observed in physical tests, such as full-field stress and deformation distributions and particle-level force chain networks.
- (4) Based on the identified fragile areas of benched slopes under ground motions, a feasible seismic retrofit method using shear pins will be developed. Different shear pin alignments will be compared in dynamic centrifugal tests to determine the most effective configuration.

Based on this work, a more precise stability evaluation method and improved design guidelines for benched slopes can be proposed, aiming to balance safety and economic considerations in engineering practice.

2.3.3 Thesis structure

Fig. 2-26 presents the research flow chart for this study. The study began with a comprehensive literature review on steep and benched slopes. Based on past research, centrifugal tests, theoretical derivations, and DEM simulations were conducted in parallel to examine the instability mechanisms of benched slopes. Based on the identified failure model, a seismic retrofit method using shear pins was subsequently proposed. Finally, practical design recommendations were provided for various site conditions.

This thesis is divided into ten parts, each of which is described in more detail below.

Chapter 1: Introduction

The research topic of benched slopes is introduced, emphasizing their functions and significance. The problems are briefly outlined, providing the motivation for this study.

Chapter 2: Literature review and methodology

Previous literature on the stability evaluation of benched slopes is reviewed to establish the scope of this study, with an emphasis on analytical, numerical, and experimental approaches. The application of these models in both 2D and 3D conditions is highlighted. A brief description of the methodology adopted for this study follows. This chapter provides foundational research methods and serves as a basis for addressing the identified research gap. The research objectives are outlined, aiming to conduct a systematic analysis of benched slope stability and provide practical recommendations for engineering applications. The overall structure of the thesis is presented at the end.

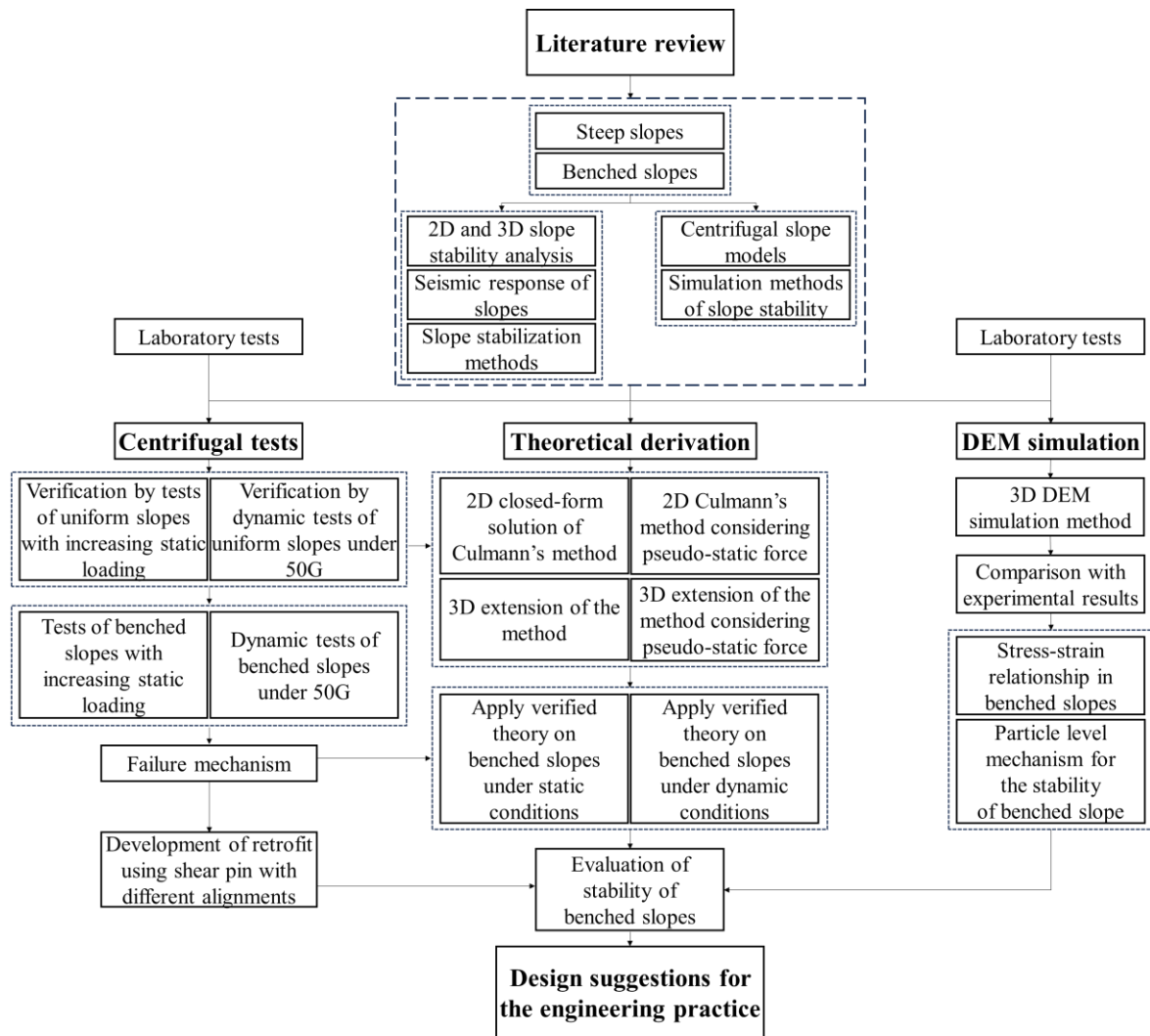


Fig. 2-26 The flow chart of this research

Chapter 3: Basic laboratory tests

The results of basic testing for material properties are presented, along with descriptions of the relevant testing devices. Constant-pressure direct shear tests and unconfined compression tests are conducted to obtain the properties of materials at varying water contents, with measured results organized statistically. Interface properties and the elastic modulus of shear pins are determined through pull-out and bending tests. Finally, the calibration process for the soil moisture sensor is described.

Chapter 4: Centrifugal modelling of steep uniform slopes under static loading conditions

A newly derived closed-form solution of Culmann's method under 2D static conditions is presented. Subsequently, Culmann's method is extended to 3D static conditions based on Rankine's theory for retaining walls. Centrifugal model tests of steep, uniform slope models are conducted to validate the proposed theories. Additionally, measured earth pressures are presented to illustrate changes in slope conditions. The necessity of applying 3D theory to laterally confined slopes under static conditions is also discussed.

Chapter 5: Centrifugal modeling of benched slopes under static loading conditions

The closed-form solution of Culmann's method under 2D static conditions is applied to benched slopes based on potential failure modes. For benched slopes with varying dimensions, the most potential sliding surface can be recognized by the lowest FS . Two sets of centrifugal model tests on benched slopes are designed, each reflecting different design concepts. The appropriate consideration of the resisting force provided by the first bench is also discussed.

Chapter 6: Evaluation of dynamic stability of steep uniform slopes by centrifugal model tests

Culmann's method is further expanded using the pseudo-static method to explain the seismic instability of uniform slopes under 2D conditions, while boundary effects are incorporated for 3D conditions. Centrifugal model tests are then conducted on uniform slope models with varying inclinations to validate the proposed theories by comparing experimental and theoretical critical horizontal seismic coefficients and failure modes. Additionally, data from installed sensors illustrate changes in slope conditions during shaking. Appropriate reduction factors for lateral resistance under dynamic conditions are suggested. The phenomenon of seismic amplification is also presented, highlighting its impact on stress-strain relationships at varying seismic intensities and heights within the slope.

Chapter 7: Evaluation of dynamic stability of benched slopes by centrifugal model tests

The seismic resistance and failure modes of benched slopes are investigated through dynamic centrifugal model tests. The pseudo-static Culmann's theory is applied to evaluate the stability of various failure modes, incorporating boundary effects and lateral resistance. Two sets of centrifugal model tests on benched slope models are conducted to validate the proposed theories. Data from installed sensors are presented to show variations in slope conditions at different locations during shaking. Experimental results are compared with the pseudo-static Culmann's method under 2D conditions. A 3D theoretical analysis is subsequently performed, using different reduction factors for various parts of the slope.

Chapter 8: Comparison of failure modes of benched slopes using centrifugal models and 3D DEM simulations

The failure mechanism of benched slopes under static and dynamic loading is investigated through simulations by the DEM. Contact properties for sphere-sphere and sphere-wall interactions are calibrated based on direct shear test results. Using these calibrated parameters, 3D DEM slope models are created at reduced dimensions to match selected cases from the centrifugal model tests. The simulations are compared with experimental results regarding critical scaling factors, failure modes, and critical input waves. Additionally, stress and displacement of the slope models are plotted, examining the effects of slope configuration and loading methods. The evolving state of the force chain network during loading is also analyzed.

Chapter 9: Impact of shear pin alignments on seismic retrofitting of benched slopes in centrifugal model tests (Tao et al., 2024)

The seismic stabilization of benched slopes with shear pins was investigated through centrifugal model tests. Various shear pin configurations on the benched slope were compared, examining the effects of pin quantity and spacing. Recorded critical horizontal seismic coefficients and observed differences in failure modes across different shear pin configurations are described. Additionally, data from installed sensors illustrate changes in slope conditions during shaking. Experimental results are compared with different theoretical assumptions regarding the mechanism by which shear pins enhance slope stability.

Chapter 10: Conclusions and future studies

The main findings and conclusions regarding the safety of benched slopes under static and dynamic loadings are summarized, and directions for future research are discussed.

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Chapter 3 Basic Laboratory tests

This chapter shows the results of laboratory tests employed to characterize the mechanical and physical characteristics of the materials adopted in centrifugal modeling.

Section 3.1 begins with an outline of the basic properties of Hiroshima sand, followed by a description of the soil moisture sensor and shear pins.

The major aim of this chapter is to obtain the strength parameters, specifically the internal friction angle (ϕ) and apparent cohesion (c), of the material at varying water contents (w) and degrees of compaction (D_c). To achieve this, elementary tests are conducted, including constant-pressure direct shear tests presented by **Section 3.2** and unconfined compression tests by **Section 3.3**.

Subsequently, **Section 3.4** and **Section 3.5** detail the measurements of interface properties and Young's modulus of the shear pins used for slope retrofit, obtained through pull-out tests and bending tests, respectively.

Section 3.6 describes the calibration process for the soil moisture sensor used to measure water content distribution within the slope models.

Finally, what obtained are summarized in **Section 3.7**.

3.1 Materials and equipment

3.1.1 Hiroshima sand

The material used in this research was called Hiroshima sand, a mountain sand taken from Kure city, Hiroshima prefecture, Japan. The raw sand that included gravel was processed by removing grains larger than 1 mm through sieving. (see **Fig. 3-1**).

Some physical properties of the sieved Hiroshima sand in dry condition are shown in **Table 3-1**. Because soil with $U_c \geq 6$ and $1 < U'_c < 3$ is considered as well graded sand (SW), and if both ranges are not met, the material is defined as poorly graded sand (SP). Thus, according to its uniformity coefficient ($U_c = 5.30$) and curvature coefficient ($U'_c = 1.25$), its classification is between SP and SW (Holtz et al., 1981).

Grain size distribution based on two tests sampled at different periods and its features observed by X-ray Micro CT are shown in **Fig. 3-2**, where two types of points represent the results of sieve analysis for material bought at different times. The composition of different batches of sand remained stable.

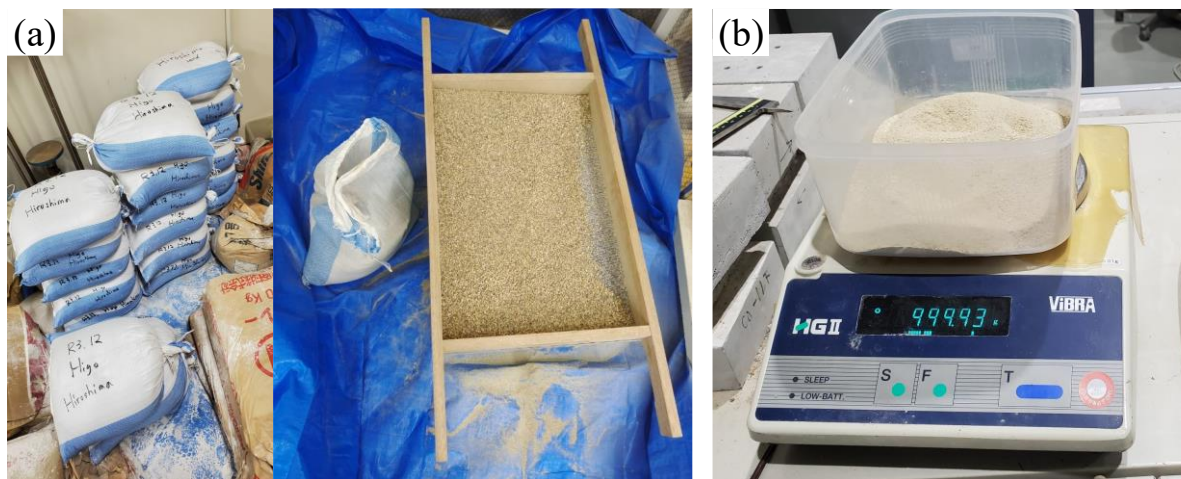


Fig. 3-1 Hiroshima sand: (a) raw sand and the sieve with 1 mm holes; (b) sieved sand

Table 3-1 Basic properties of the sieved Hiroshima sand

Specific gravity (G_s)	2.633
Average particle diameter (D_{50})	0.33~0.34 mm
Uniformity coefficient (U_c)	5.18~5.42
Curvature coefficient (U'_c)	1.20~1.29
Maximum void ratio (e_{max})	1.006
Minimum void ratio (e_{min})	0.444
Fines content (FC)	9.2%
Optimum water content (w_{opt})	11.2 %
Maximum dry density (ρ_{dmax})	1747 kg/m ³

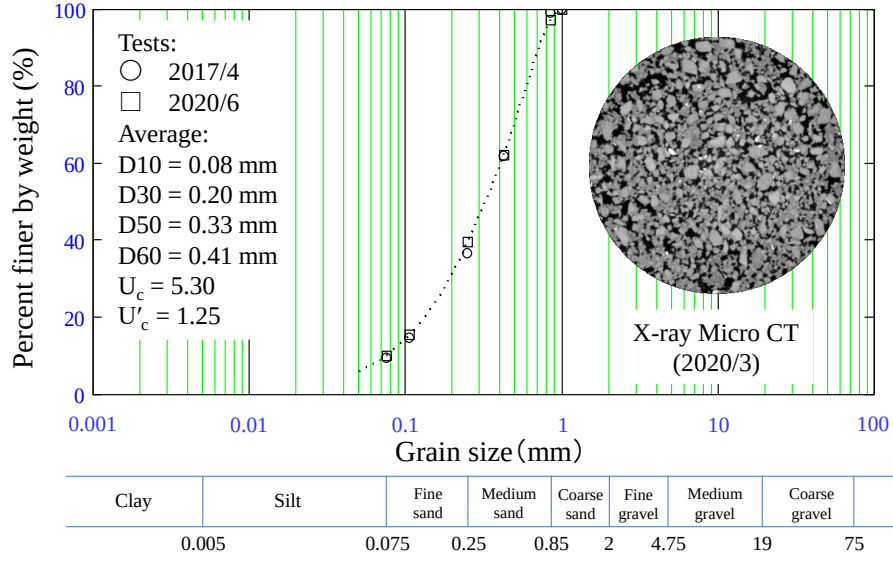


Fig. 3-2 Hiroshima sand after removal of grain size larger than 1 mm

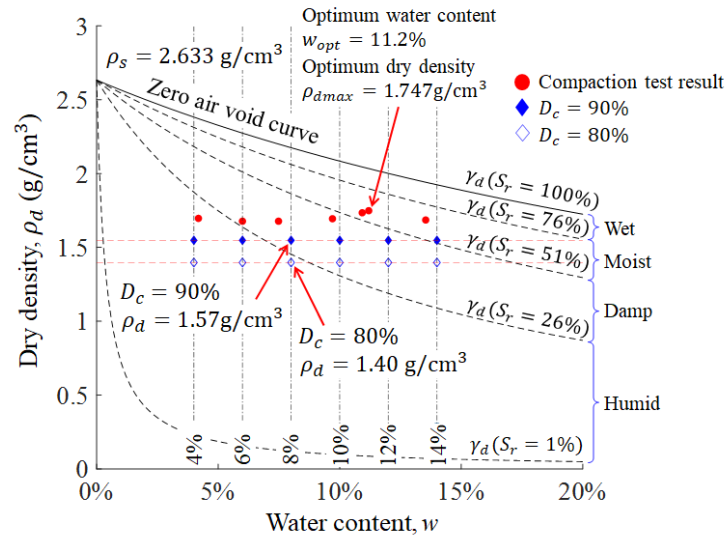


Fig. 3-3 Compaction curve of Hiroshima sand and saturation lines under various soil conditions

The compaction property of this sand is illustrated in **Fig. 3-3**. Since this sand is not well graded and contains low fines content, the compaction curve was not obvious. As indicated by the round points, a maximum dry density (ρ_{dmax}) of 1747 kg/m³ and a corresponding optimum moisture content (w_{opt}) of 11.2% was adopted from one batch of tests. According to Terzaghi et al. (1996), the degree of saturations, S_r , can describe humid, damp, moist and wet conditions of soil. Therefore, lines representing different S_r values were imposed on the compaction curve to classify various regions of the compacted sand condition. Preliminary trials indicate that Hiroshima sand is well-suited for slope stability modeling due to its adjustable strength based on water content and compaction degree. Using this material, the slope model can maintain stability even at a high inclination. To observe slope failure in static loading

condition under centrifugal acceleration less than 100G, the slope model constructed by tamping Hiroshima sand to an 80% compaction degree, with a water content of 8%, which is considered humid sand (as denoted by the arrows in the figure). For dynamic loading conditions under centrifugal acceleration of 50G, 90% compaction degree with a water content of 8%, which is considered damp sand, were adopted. The outcomes of soil compaction tests under desired conditions are summarized in **Table 3-2** and **Table 3-3**.

Table 3-2 Basic properties of Hiroshima sand ($D_c=80\%$)

Water content (w)	8.0	%
Total density (ρ_t)	1510	kg/m ³
Dry density (ρ_d)	1398	kg/m ³
Void ratio (e)	0.884	
Degree of saturation (S_r)	24	%

Table 3-3 Basic properties of Hiroshima sand ($D_c=90\%$)

Water content (w)	8.0	%
Total density (ρ_t)	1698	kg/m ³
Dry density (ρ_d)	1572	kg/m ³
Void ratio (e)	0.675	
Degree of saturation (S_r)	31	%

The procedure of measuring shear strength properties of these two conditions of compacted unsaturated material will be described in **Section 3.2** and **Section 3.3**.

3.1.2 Shear pins

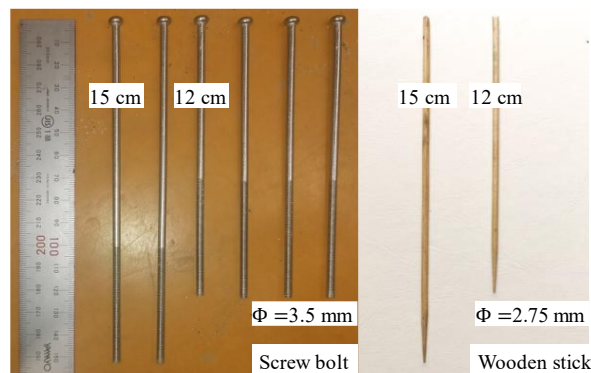


Fig. 3-4 Two types of shear pins used for slope reinforcement

To develop an economical slope reinforcement method, shear pins were inserted in model slopes. Two types of shear pins made of wood and steel were used (see **Fig. 3-4**), both having two lengths (15

cm and 12 cm). They all have a round cross-section, with a diameter of 2.75 mm for wooden sticks and 3.5 mm for steel screw bolts.

The testing process for determining the interface properties between the shear pins and Hiroshima sand is detailed in **Section 3.4**, while the measurement of the shear pins' Young's moduli is presented in **Section 3.5**.

3.1.3 Soil moisture sensor

To measure the water content inside slope models, the soil moisture sensor, type GTI-S50, made by GRI (Geo-research institute), was used in this study (see **Fig. 3-5**).

The head of the moisture sensor is connected to a junction box, which is subsequently linked to the datalogger via an attenuator cable, type CR-4010. The calibration process for this sensor will be detailed in **Section 3.6**.

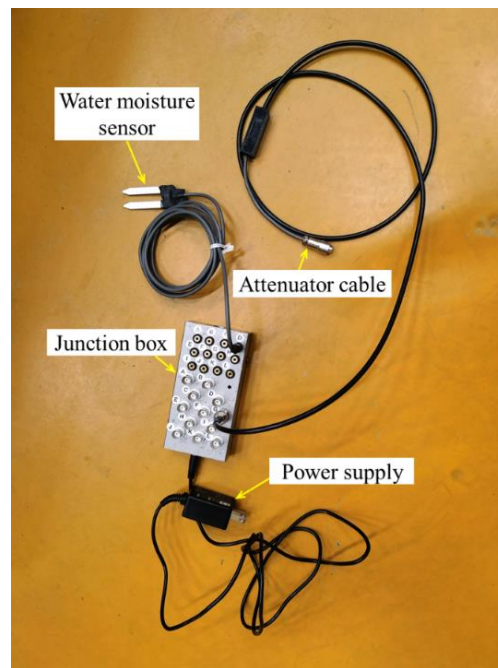


Fig. 3-5 Equipment for measuring water content

3.2 Constant-pressure direct shear test

3.2.1 Apparatus of direct shear test

To obtain the shear properties of Hiroshima sand, constant-pressure direct shear tests (DST) were employed, with testing machine GTI-DS 6010. A photograph of this device is shown in **Fig. 3-6**, and a schematic figure of it is presented in **Fig. 3-7**. The shear box includes two parts, an upper box made of polycarbonate and a steel bottom box. The upper box can be fixed by bolts, with a horizontally sliding free part hanging below the reaction plate.

In the consolidation stage, the vertical loading is applied on the lower box from downwards by a loading piston, which is driven by compressed air in the cylinder. During the shear process, the

magnitude of vertical loading fluctuates in a controlled range, which is controlled by a servo system. The shear force exerted on the upper box is conducted by the DD motor from the right-hand side with a constant driving speed.

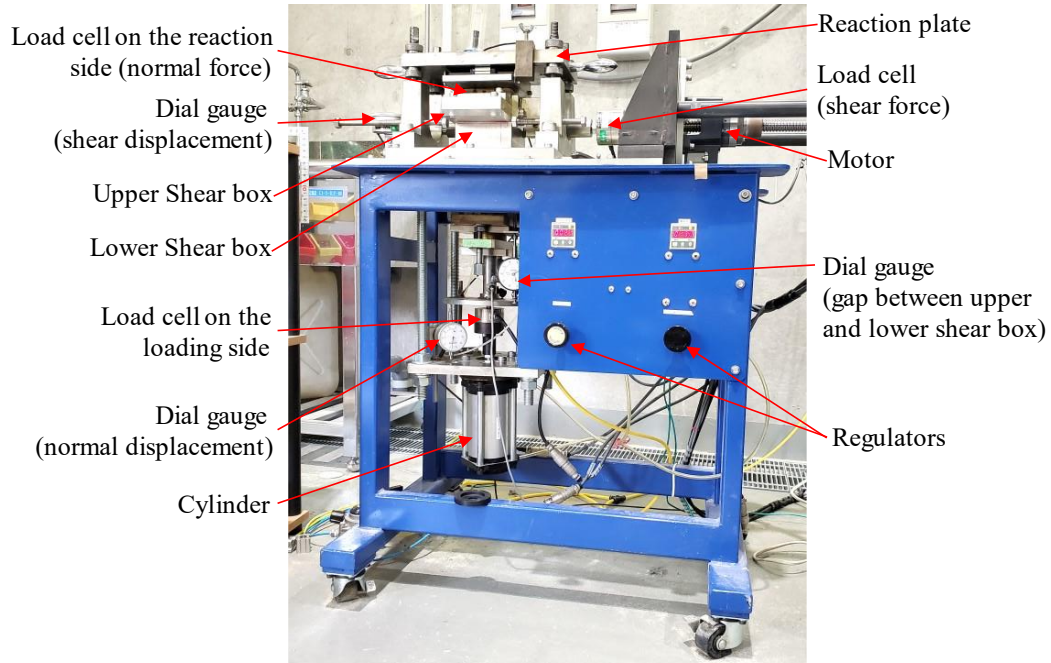


Fig. 3-6 Direct shear testing machine

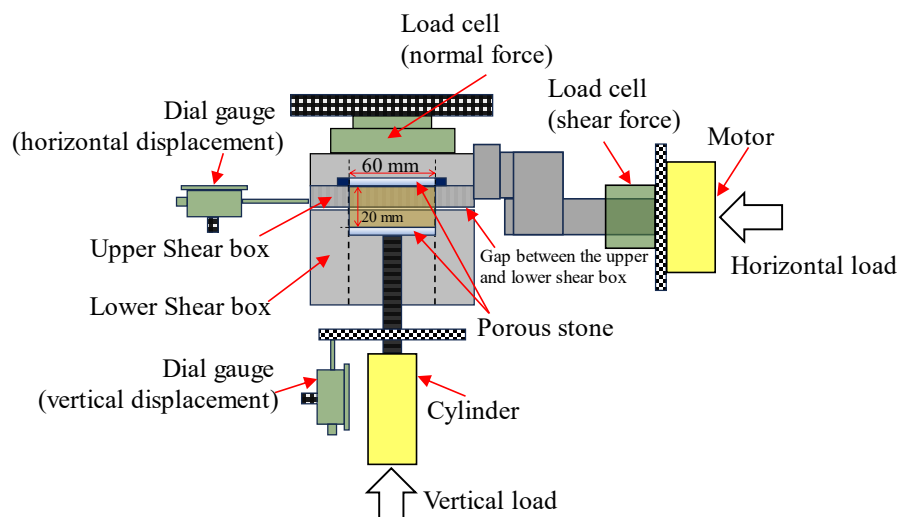


Fig. 3-7 Schematic figure of the direct shear testing machine

The displacements during the shearing process are measured by two dial gauges. The shear force is recorded by a load transducer adjacent to the motor. Along vertical direction, there are two load cells in the loading side and reaction side, respectively. The pressure measured from the load transducer on the reaction side is taken as the normal pressure for the specimen.

3.2.2 Test procedure

Following JGS 0561 “Method for consolidated constant pressure direct box shear test on soils” (Japanese-geotechnical-society, 2020), two groups of constant-pressure direct shear tests were conducted for sand specimens with two different compaction degrees ($D_c=80\%$, 90%). They were performed with a shear displacement rate of 0.2 mm/min . The tests stopped when the upper shear box reached a shear displacement of 7 mm . The cylindrical shear box for specimen had a diameter of 60 mm and a height of 20 mm . The clearance between the upper and lower shear boxes was set to 0.2 mm , which is less than the D_{50} of Hiroshima sand. The measured maximum shear stress indicates peak strength, while the ending shear stress represents the residual strength.

In centrifugal model tests, though attempts were made to minimize evaporation to retain water content, the variation with depth cannot be completely suppressed due to soil water retention and percolation under gravitation. Therefore, not only strengths of the specimen with a target water content of 8% , but its neighboring water contents of 6% and 10% were also examined to explain the effect of suction induced by the change in water content. For each testing group, the dry sand was blended with water uniformly to reach different water contents ($w=0\%$, 6% , 8% , 10%). The weight of soil needed to make one specimen can be calculated by the volume of the shear cylinder, the wet density of the material and the objective compaction degree. Then, the soil was divided into four portions, filled into the shear cylinder and tamped by a hammer to achieve a uniform state layer by layer. The interface between each two layers of sand was scratched to ensure that different layers combined tightly without discontinuity. For one water content with one compaction degree, five specimens were tested under normal pressure 40 kPa , 60 kPa , 80 kPa , 100 kPa and 120 kPa , respectively.

3.2.3 Testing results

The testing results are as follows. **Fig. 3-8** to **Fig. 3-11** are from $D_c=80\%$, **Fig. 3-12** to **Fig. 3-15** are from $D_c=90\%$. They present the relationship among shear stress (τ), shear strain (γ), normal stress (σ) and normal strain (ϵ), with varying water content (w).

The measured peak-strength and residual-strength by the DST are summarized in **Table 3-4**. The linear regression of the results, terminated at a fixed shear strain of 0.35 , does not precisely represent the critical-state strength where the dilatancy ratio is zero.

For the sample with D_c of 80% , while contraction was mainly observed in dry samples, dilation was mainly observed in unsaturated samples, especially low levels of normal stress. Due to the low level of compaction, the peak strengths observed in the stress-strain curves between τ and γ were not significantly higher than the residual strengths. Linear regression was performed to determine the intercept value and slope according to the Mohr-Coulomb criteria from both residual and peak strengths at each moisture content, which is listed in **Table 3-5**.

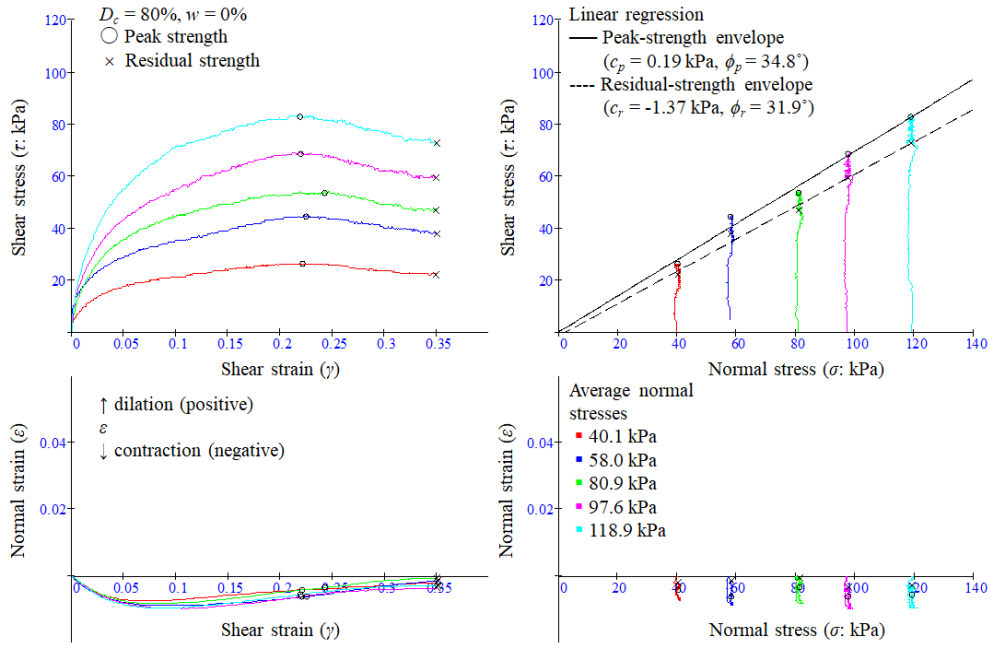


Fig. 3-8 Constant-pressure DST ($D_c=80\%$, $w=0\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

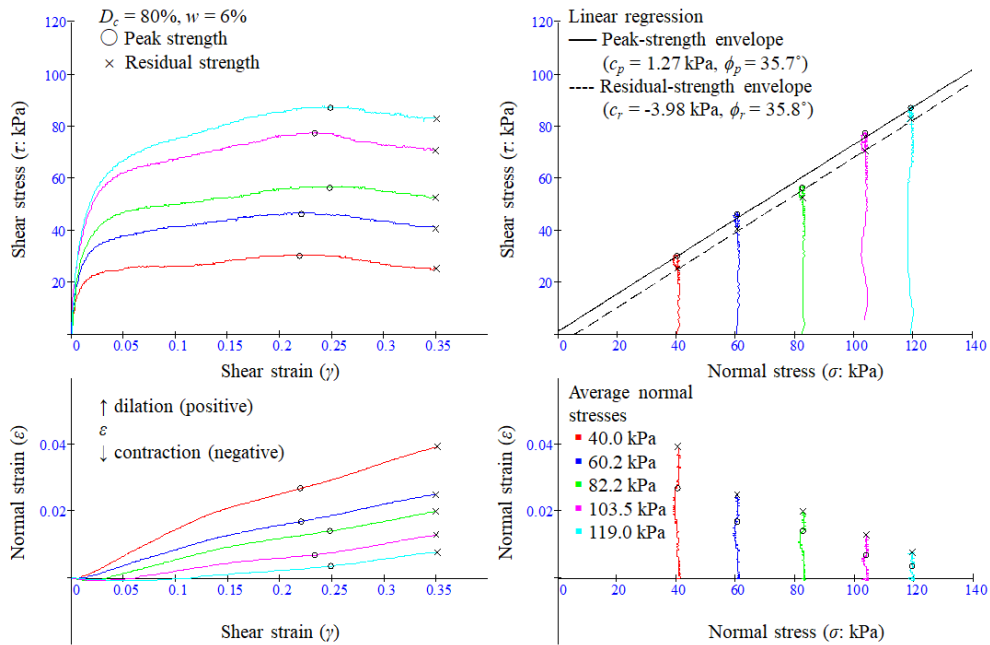


Fig. 3-9 Constant-pressure DST ($D_c=80\%$, $w=6\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

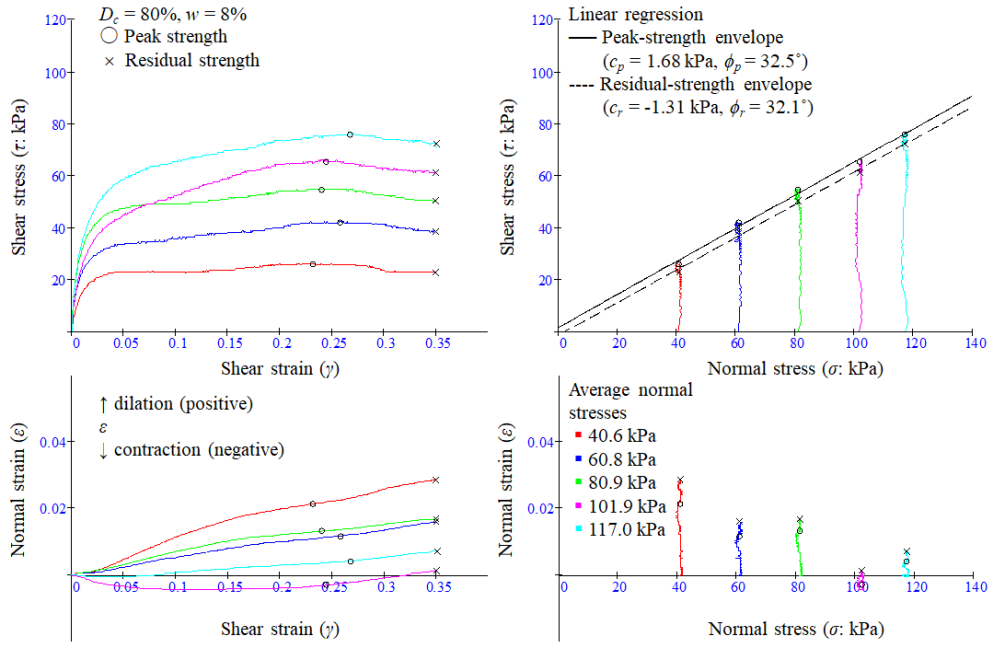


Fig. 3-10 Constant-pressure DST ($D_c=80\%$, $w=8\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

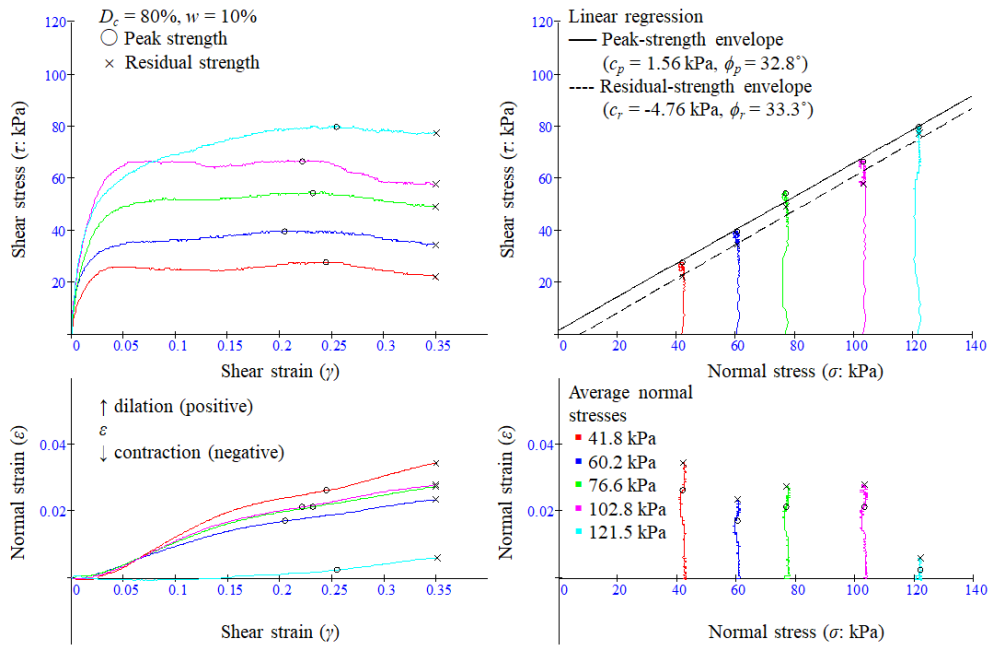


Fig. 3-11 Constant-pressure DST ($D_c=80\%$, $w=10\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

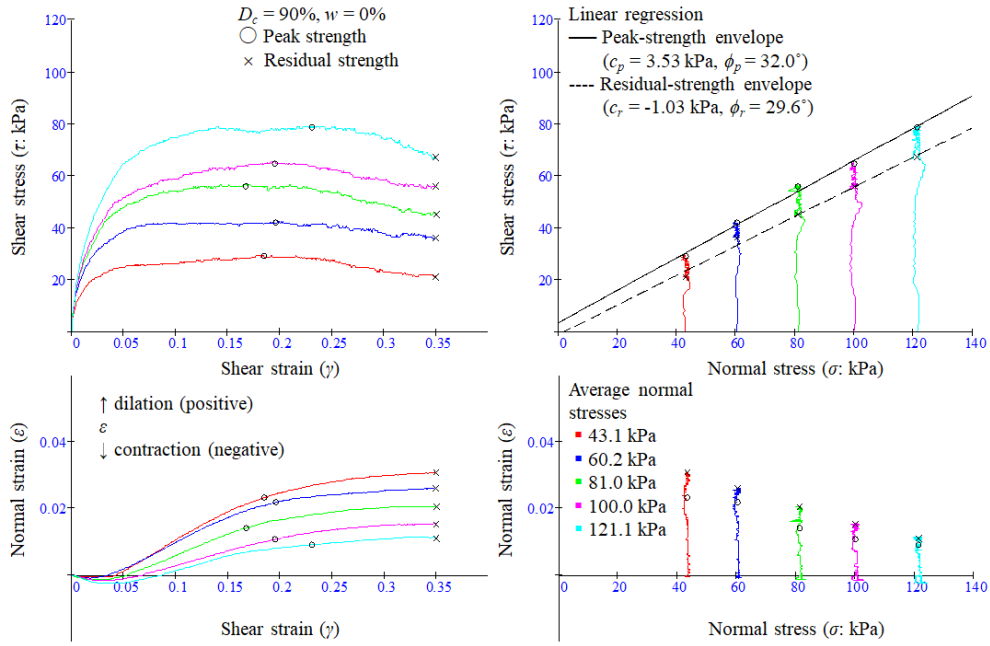


Fig. 3-12 Constant-pressure DST ($D_c=90\%$, $w=0\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

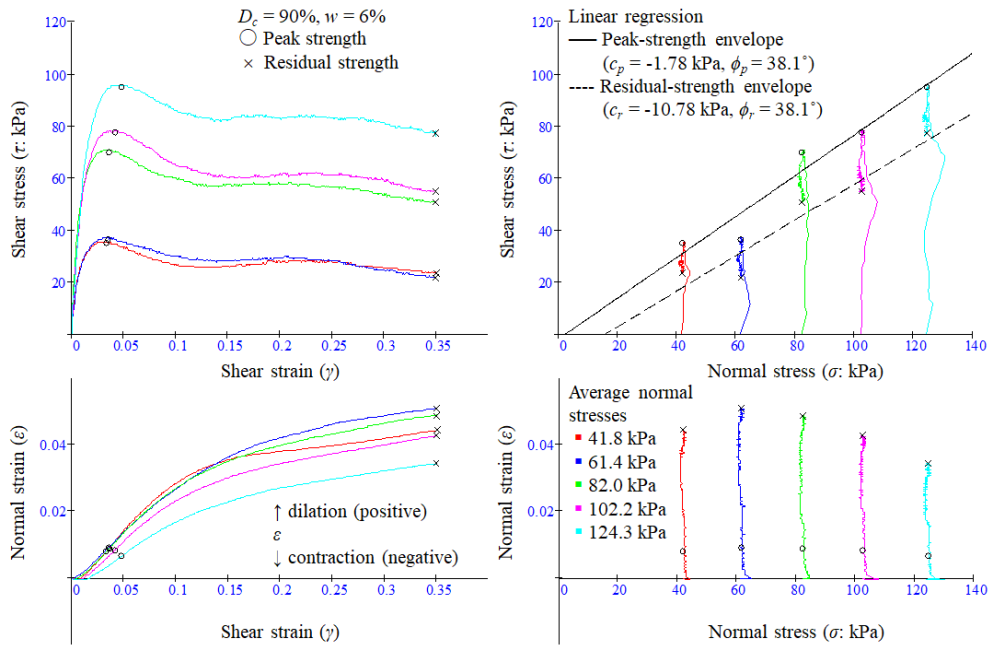


Fig. 3-13 Constant-pressure DST ($D_c=90\%$, $w=6\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

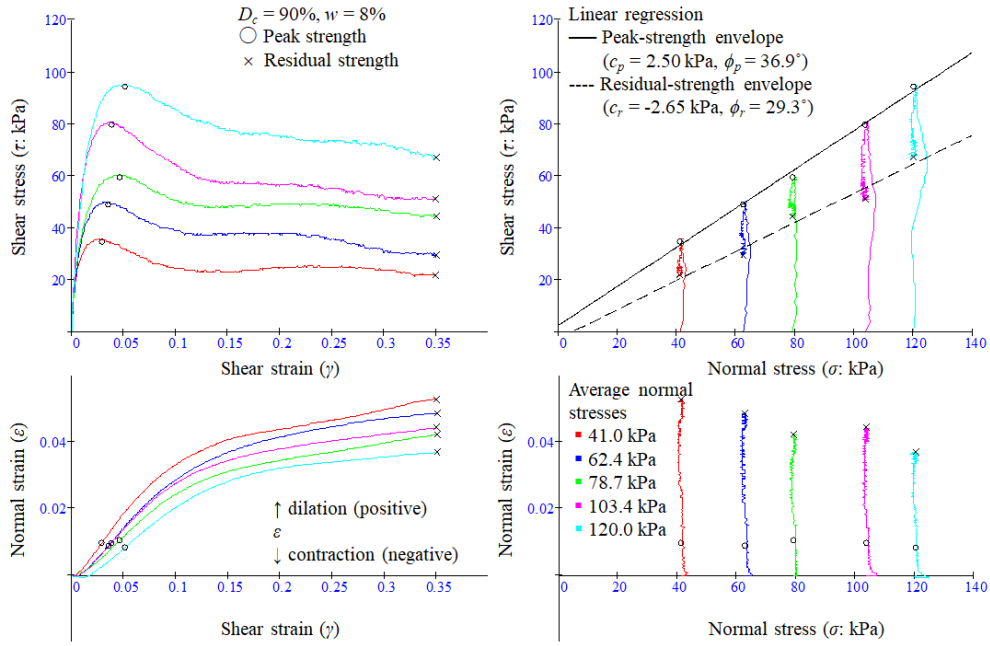


Fig. 3-14 Constant-pressure DST ($D_c=90\%$, $w=8\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

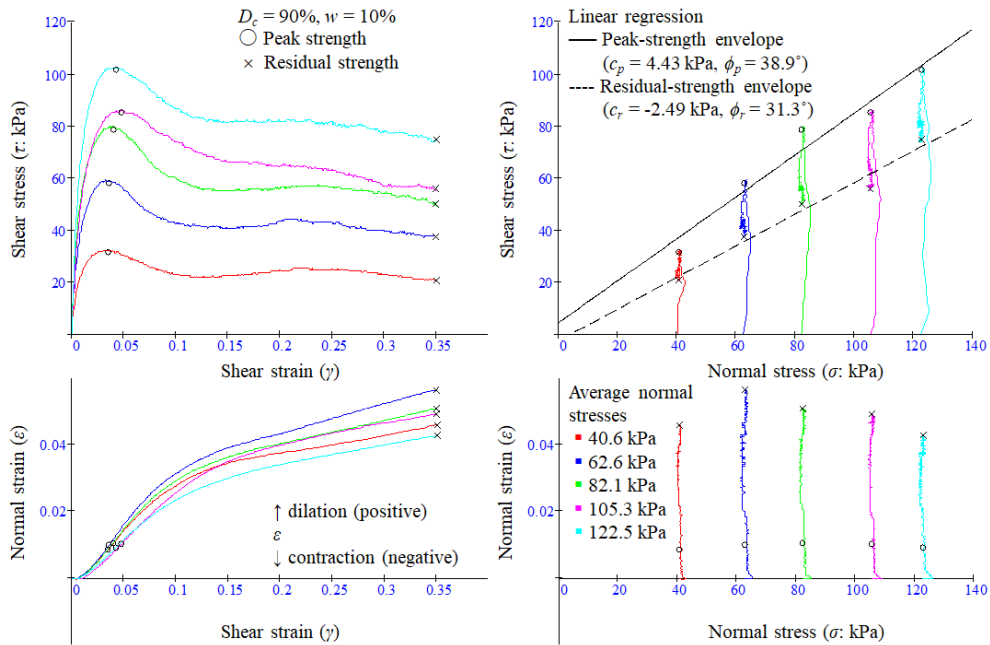


Fig. 3-15 Constant-pressure DST ($D_c=90\%$, $w=10\%$): (a) shear stress vs. shear strain; (b) shear stress vs. normal stress; (c) normal strain vs. shear strain; (d) normal strain vs. normal stress

The narrow range of friction angles, found between 32° - 36° for both peak and residual strengths,

implies that except for the dry condition, there is no clear difference in the slope of the failure envelope across different water content between 6-10%; hence, both failure envelopes appear relatively parallel. Nevertheless, the intercepts were found to be slightly positive for the peak-strength envelope and slightly negative for the residual-strength envelope. As a negative intercept cannot be accepted as cohesion, it is necessary to consider the error range for measurements. The apparatus employed for testing has a limitation in that the applied pressure cannot be below 40 kPa. The negative intercept obtained from the residual strength is a result of this limitation when cohesion is very small. Due to data variability, standard errors for the intercept and slope were included.

The method to calculate the standard error is shown as follows. **Eq. (3-1)** is the result of a simple regression, where b_0 and b_1 are the regression intercept and slope, respectively. $\hat{Y}_t$ is the prediction for a X_t . The error, e_t , that the regression made is the difference between the real Y_t and the prediction, shown in **Eq. (3-2)**. The standard error of the model, s , is calculated by **Eq. (3-3)**. Finally, standard errors for the intercept and the slope can be obtained from **Eq. (3-4)** and **Eq. (3-5)**, respectively, where n is the sample size.

$$\hat{Y}_t = b_0 + b_1 X_t \quad (3-1)$$

$$e_t = Y_t - \hat{Y}_t \quad (3-2)$$

$$s = \sqrt{\frac{1}{n-2} \sum_{t=1}^n e_t^2} \quad (3-3)$$

$$SE_{b_0} = \frac{s}{\sqrt{n}} \sqrt{1 + \frac{\text{mean}(X)}{\text{VAR.P}(X)}} \quad (3-4)$$

$$SE_{b_1} = \frac{s}{\sqrt{n}} \frac{1}{\text{STDEV.P}(X)} \quad (3-5)$$

Table 3-4 Strength properties of Hiroshima sand with degrees of compaction and water content

Degree of compaction (D_c): %	Water content (w): %	Peak friction angle (ϕ_p): °	Residual friction angle (ϕ_r): °	Peak cohesion (c_p): kPa	Residual cohesion (c_r): kPa
80	0	34.8	31.9	0.19	-1.37
	6	35.7	35.8	1.26	-3.98
	8	32.5	32.1	1.68	-1.31
	10	32.8	33.3	1.56	-4.76
90	0	32.0	29.6	3.53	-1.03
	6	38.1	34.4	-1.78	-10.78
	8	36.9	29.3	2.50	-2.65
	10	38.9	31.3	4.43	-2.49

Table 3-5 Linear regression of failure envelopes obtained from direct shear tests (DST) of Hiroshima

sand with 80% compaction degree and 0%-10% water contents			
Water content (w): %	Slope $\pm$ standard error	Intercept $\pm$ standard error: kN/m ²	Failure envelopes
0	$\tan(34.8^\circ) \pm 0.048$	0.19 ± 4.02	Peak strength
6	$\tan(35.7^\circ) \pm 0.041$	1.26 ± 3.54	
8	$\tan(32.5^\circ) \pm 0.026$	1.68 ± 2.18	
10	$\tan(32.8^\circ) \pm 0.034$	1.56 ± 2.92	
0	$\tan(31.9^\circ) \pm 0.038$	-1.37 ± 3.21	Residual strength
6	$\tan(35.8^\circ) \pm 0.029$	-3.98 ± 2.53	
8	$\tan(32.1^\circ) \pm 0.025$	-1.31 ± 2.12	
10	$\tan(33.3^\circ) \pm 0.061$	-4.76 ± 5.19	

For samples with D_c of 90%, dilation was observed in all samples and its level was more pronounced than samples with D_c of 80%. Except for the dry sample, the stress-strain curves have clear early peaks, and there is a distinct difference between the peak and the residual strengths. Linear regression results of the intercept and slope according are summarized in **Table 3-6**. The peak friction angles are between 32°-39°, which are higher than the previous group, while the residual friction angles are between 29°-35°, which are slightly lower than the previous group.

Table 3-6 Linear regression of failure envelopes obtained from direct shear tests (DST) of Hiroshima sand with 90% compaction degree and 0%-10% water contents

Water content (w): %	Slope $\pm$ standard error	Intercept $\pm$ standard error: kN/m ²	Failure envelopes
0	$\tan(32.0^\circ) \pm 0.027$	3.53 ± 2.27	Peak strength
6	$\tan(38.1^\circ) \pm 0.116$	-1.78 ± 10.11	
8	$\tan(36.9^\circ) \pm 0.029$	2.50 ± 2.52	
10	$\tan(38.9^\circ) \pm 0.098$	4.43 ± 8.61	
0	$\tan(29.6^\circ) \pm 0.037$	-1.03 ± 3.18	Residual strength
6	$\tan(34.4^\circ) \pm 0.120$	-10.78 ± 10.49	
8	$\tan(29.3^\circ) \pm 0.060$	-2.65 ± 5.11	
10	$\tan(31.3^\circ) \pm 0.064$	-2.49 ± 5.63	

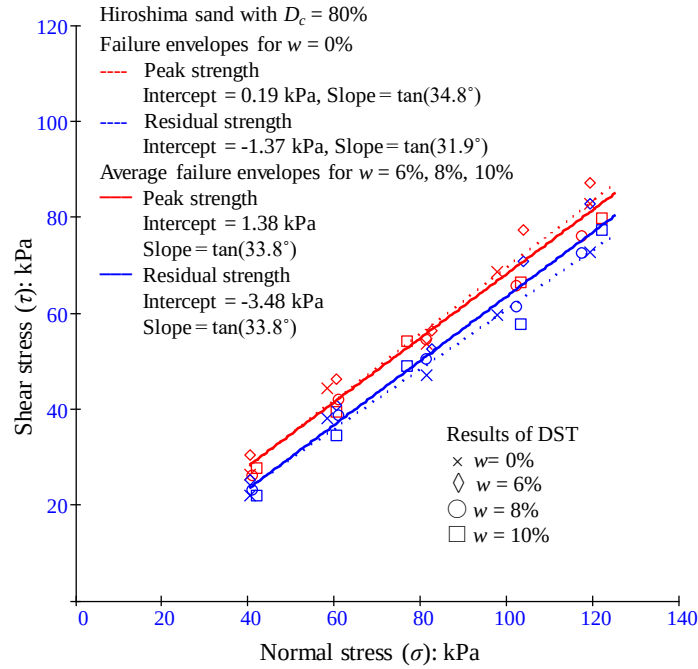


Fig. 3-16 Failure envelopes for peak and residual strength of Hiroshima sand with 80% compaction degree

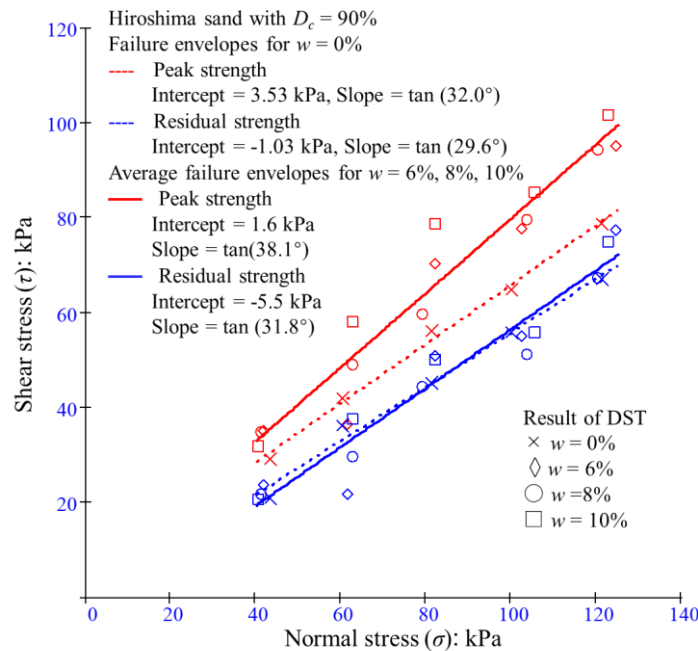


Fig. 3-17 Failure envelopes for peak and residual strength of Hiroshima sand with 90% compaction degree

Though the impact of suction on shear properties is significant, suction-controlled apparatus was not employed here. Therefore, the results of DST under dry condition ($w = 0\%$), representing a case of zero

suction, were plotted together with those under unsaturated conditions ($w = 6, 8, 10\%$) for comparison, as shown in following figures. For the samples with D_c of 80% (see **Fig. 3-16**), the average slopes of the peak-strength and residual-strength failure envelopes under dry condition were $\tan(34.8^\circ)$ and $\tan(31.9^\circ)$, respectively, and those under unsaturated conditions were approximately $\tan(33.8^\circ)$. However, variation of intercepts was observed for peak and residual strengths under dry and unsaturated conditions. Hence, the suction of compacted humid sand used in this study mainly affects the apparent cohesion rather than the friction angle. For the samples with D_c of 90% (see **Fig. 3-17**), the average failure envelope inclination for unsaturated samples is higher than other conditions; thus, the suction plays a role in both the cohesion and the internal friction angle for damp sand.

The shear strength of unsaturated soils typically decreases from a peak to a residual value, exhibiting strain-softening behavior under large deformations (Yang & Vanapalli, 2023). Yang and Vanapalli (2018) analyzed the stability of a slope and presented the variation in the factor of safety by associating it with peak and residual strength parameters for different matric suction values. For a rational slope design, residual strength should be used for a conservative estimate of stability, while peak strength is applied for analyzing the instability of slopes that are intact and have not experienced significant movement or disturbance. Hence, in this research, the mean peak-strength of Hiroshima sand with 80% and 90% compaction degree with 6%~10% water contents based on DST (see **Table 3-7** and **Table 3-8**), are applied to centrifugal slope models, as these models represent newly constructed slopes that have not yet undergone any form of shearing.

Table 3-7 Means and standard errors for peak-strength parameters obtained from DST of Hiroshima sand with 80% compaction degree and 6%~10% water contents

Shear strength parameters	Mean	Standard error
Friction angle (ϕ_p)	33.8°	1.9°
Apparent cohesion (c_p)	1.38 kN/m ²	2.88 kN/m ²

Table 3-8 Means and standard errors for peak-strength parameters obtained from DST of Hiroshima sand with 90% compaction degree and 6%~10% water contents

Shear strength parameters	Mean	Standard error
Friction angle (ϕ_p)	38.1°	3.1°
Apparent cohesion (c_p)	1.6 kN/m ²	4.8 kN/m ²

3.3 Unconfined compression test

3.3.1 Apparatus of unconfined compression test

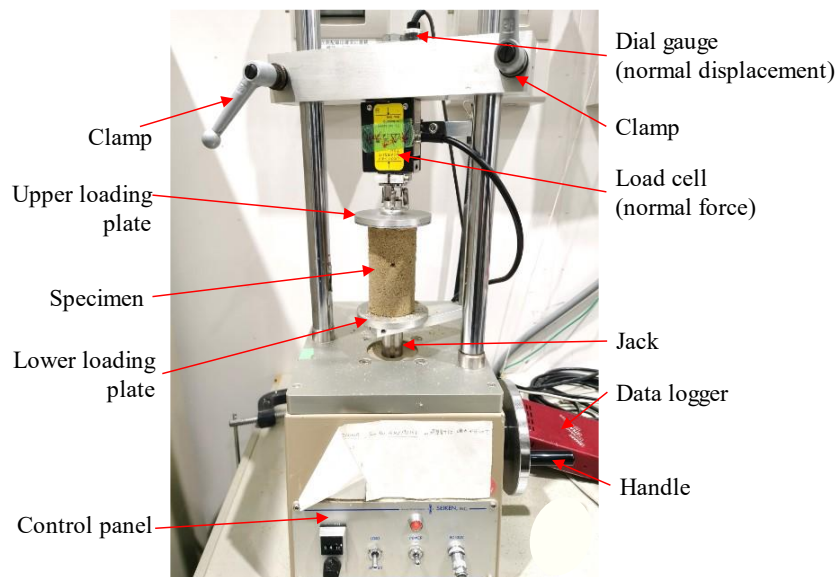


Fig. 3-18 Unconfined compression testing machine

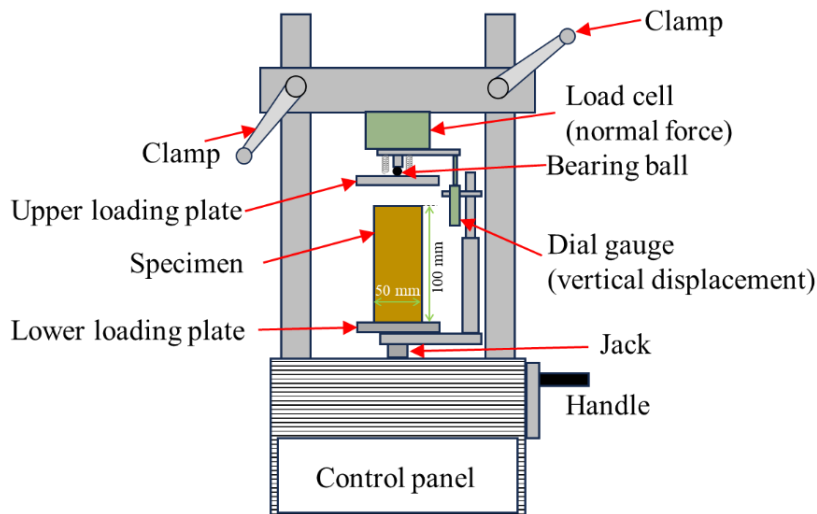


Fig. 3-19 Schematic diagram of the unconfined compression testing machine

As the direct shear tests (DST) did not provide a clear relationship between the cohesion of Hiroshima sand and water content, unconfined compression tests (UCT) were conducted for verification. The device, manufactured by SEIKEN, is illustrated in **Fig. 3-18**, and the schematic diagram is shown in **Fig. 3-19**. Before compression, a clamp is used to change the position of the upper loading plate to make it just contact the specimen column.

During the compression, the soil column is loaded by the lower plate, which is connected to a jack moving at a constant velocity controlled via the panel. The compression load applied to the specimen is recorded by a load transducer fixed above the upper loading plate, while changes in specimen height

are recorded by a dial gauge.

3.3.2 Test procedure

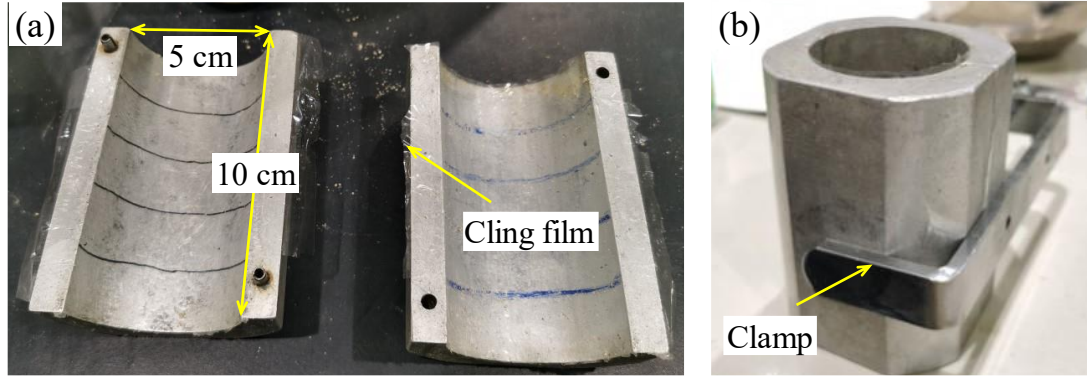


Fig. 3-20 Divisible steel hollow cylindrical container: (a) dimension and pretreatment of the inner face of the container; (b) combined container by a clamp

The UCT was conducted based on JGS 0511 “Method for Unconfined Compression Test of Soils” (Japanese-geotechnical-society, 2019). To prepare the specimen of unsaturated Hiroshima sand, a divisible steel hollow cylindrical container was used (see **Fig. 3-20**). The container’s inner cylindrical space had a height of 100 mm and a diameter of 50 mm. It was divided evenly into five layers for the soil compaction process (see **Fig. 3-20 (a)**). The two halves of the container were then assembled using a clamp (see **Fig. 3-20 (b)**).

Two compaction degrees ($D_c=80\%, 90\%$) were used in this test with varying water content ($w=0\%, 4\%, 6\%, 8\%, 10\%, 12\%$). The weight of soil needed to fill one layer can be calculated from one layer’s volume, the material’s wet density and the objective compaction degree. The material is poured into the cylinder and tamped by a hammer to achieve a uniform state. The upper surface of each layer of material was scratched to ensure that layers combined tightly. After finishing the compaction of all five layers, the container with soil was transferred on the lower loading plate of the apparatus (see **Fig. 3-18**). The clamp was then loosened and the two half container components were taken away carefully, leaving the specimen standing by its weight. Subsequently, the height of upper loading plate was adjusted to make it just touch the specimen without applying any force.

In the compression process, the lower loading plate was lifted by the jack with loading rate maintained at 1 mm/min. The normal load was recorded by a load cell. The displacement was measured by a dial gauge. The compression was stopped when the force decreased to 70% of the peak value.

3.3.3 Test results

For each w , two cases were conducted, except for $w=8\%$, for which three cases were conducted. The axial stress (σ_a) and axial strain (ε_a) for all cases, varying D_c and w , and the curve of unconfined

compressive strength (UCS) with respect to water content are as shown as **Fig. 3-21** and **Fig. 3-22**.

As presented in **Fig. 3-21 (f)**, for $D_c=80\%$, the UCS of Hiroshima sand under 80% compaction degree increases with w up to the optimum water content (w_{opt}). For $D_c=90\%$, the UCS increases firstly, then decreases after $w>8\%$ (see **Fig. 3-22 (f)**). The UCS for both D_c is listed in **Table 3-9** and **Table 3-10**.

According to **Fig. 3-23**, the photos taken at failure during UCT for all tests indicate that soil samples have undergone shear deformation and reached shear failure at an axial strain as small as around 2%, leading to the progressive generation of inclined shear bands, with slightly barreling and axial splitting. Although a drop in axial stress after the peak was typically observed, this does not represent strain-softening behavior but rather a change in the loading transmission mode, as the sample could not maintain the condition of uniaxial compression due to the instability and the rotation of the top cap at failure.

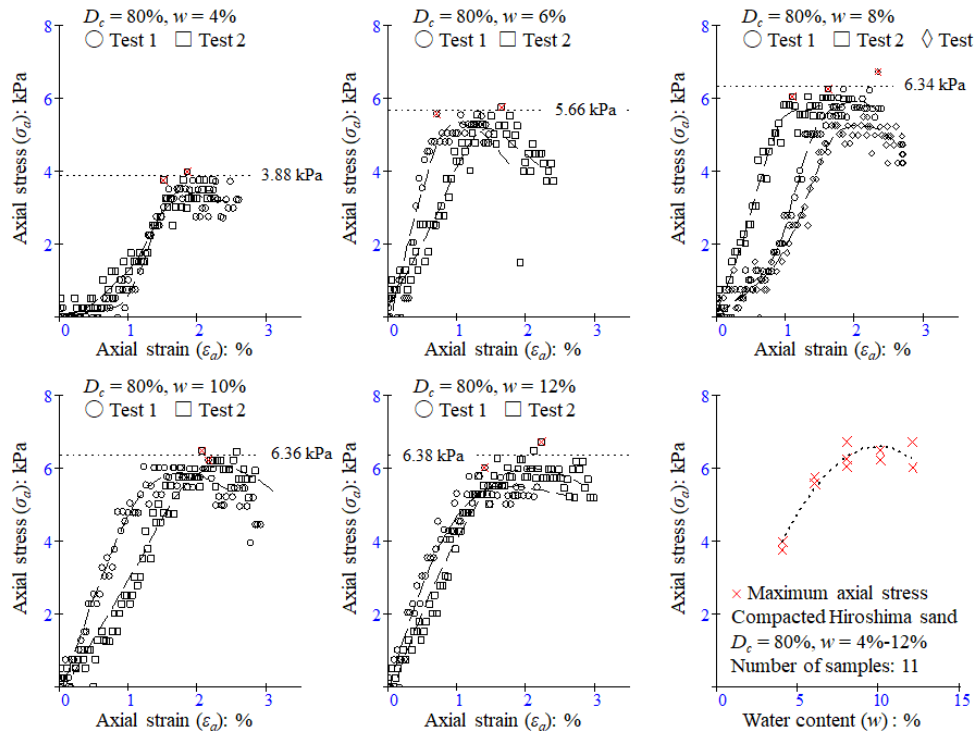


Fig. 3-21 UCT of Hiroshima sand with $D_c=80\%$: (a) (b) (c) (d) (e) stress-strain relationship; (f) unconfined compressive strength vs. water content

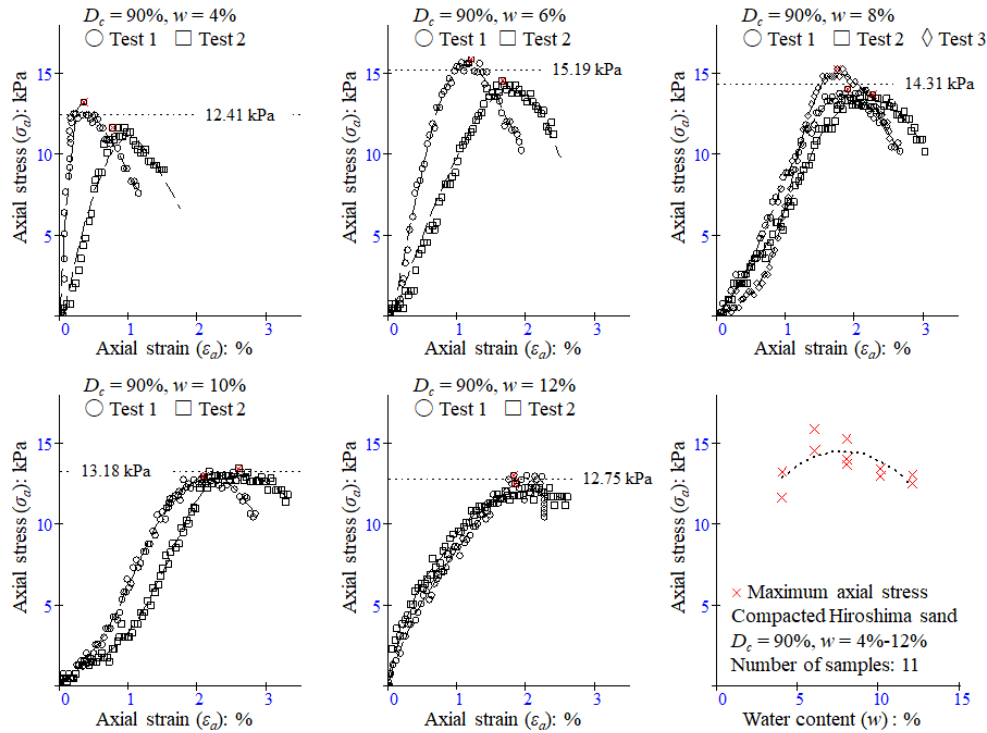


Fig. 3-22 UCT of Hiroshima sand with $D_c=90\%$: (a) (b) (c) (d) (e) stress-strain relationship; (f) unconfined compressive strength vs. water content

Table 3-9 Unconfined compressive strength (UCS) of Hiroshima sand ($D_c = 80\%$)

Water content (w): %	Degree of saturation (S_r): %	Average unconfined compressive strength (σ_c): kg/m ²
4	12	3.88
6	18	5.66
8	24	6.34
10	30	6.36
12	36	6.38

Table 3-10 Unconfined compressive strength (UCS) of Hiroshima sand ($D_c = 90\%$)

Water content (w): %	Degree of saturation (S_r): %	Average unconfined compressive strength (σ_c): kg/m ²
4	16	12.41
6	24	15.19
8	31	14.31
10	39	13.18
12	49	12.75

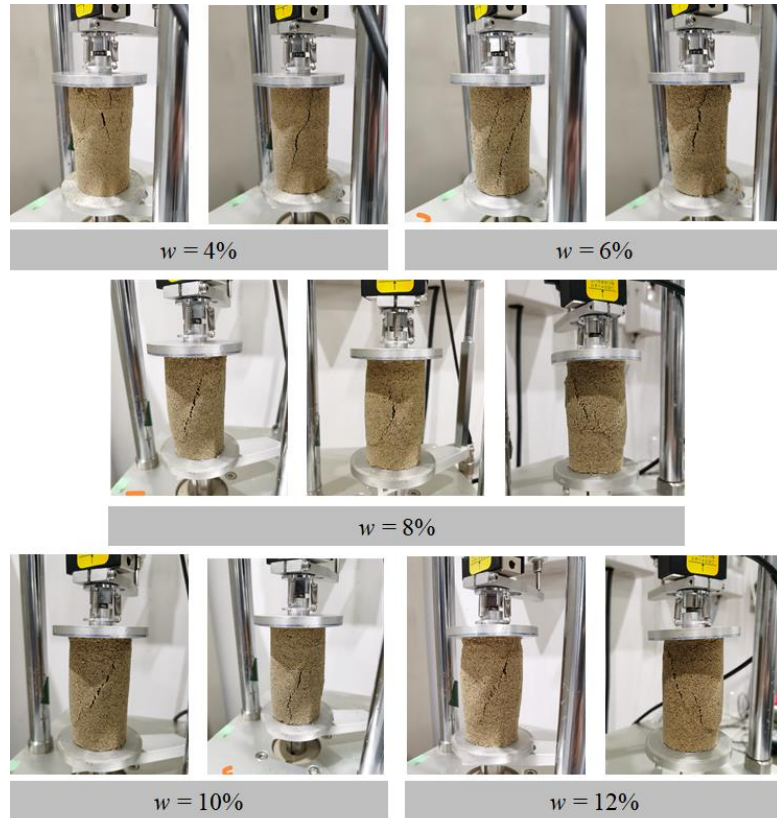


Fig. 3-23 Failure observed in unconfined compression tests for Hiroshima sand with 80% compaction degree and 4%~12% water contents

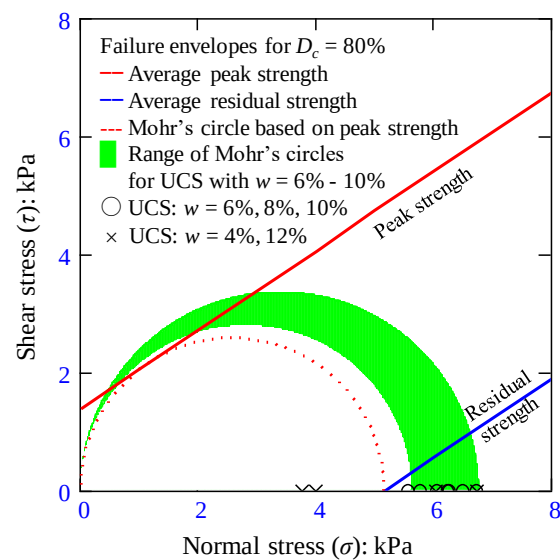


Fig. 3-24 Range of Mohr's circles presenting unconfined compressive strength (σ_c) plotted with the peak-strength and residual-strength envelopes of Hiroshima sand with 80% compaction degree

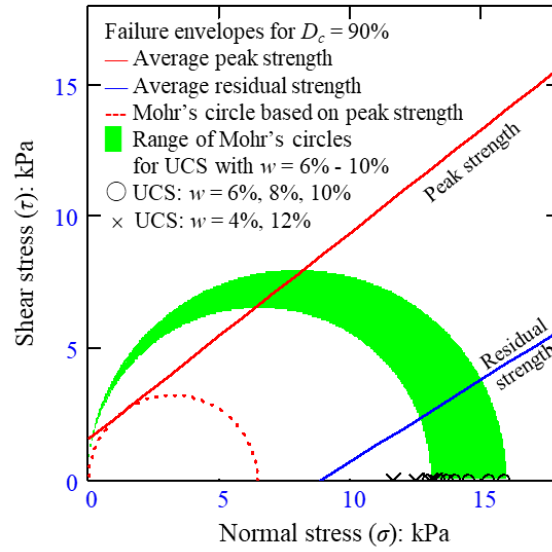


Fig. 3-25 Range of Mohr's circles presenting unconfined compressive strength (σ_c) plotted with the peak-strength and residual-strength envelopes of Hiroshima sand with 90% compaction degree

$$\sigma_c = 2c \cdot \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad (3-6)$$

The range of Mohr's circles in total stress, based on UCT, is plotted with the peak-strength and residual-strength envelopes, based on DST, in **Fig. 3-24** and **Fig. 3-25** for different compaction degrees. The dashed circle represents the unconfined Mohr's circle derived through the peak strength envelope from the DST. This is compared to the circle obtained from the UCT, depicted as the green range. Referring to **Eq. (3-6)**, the value of unconfined compressive strength (σ_c) calculated using $c = c_p$ and $\phi = \phi_p$ is 5.15 kPa for 80% compaction degree and 6.43 kPa for 90% compaction degree, which is roughly 84% and 45% of the average σ_c obtained for w of 6, 8, 10%, as summarized in **Table 3-9** and **Table 3-10**. Given the material's low cohesion, variability may arise from sample preparation and device manipulation. However, this difference is considered acceptable. The comparison confirms that σ_c obtained from UCT is consistent with the peak strength parameters obtained from DST.

3.4 Pull-out test

3.4.1 Apparatus of pull-out experiment

The pull-out experiment was conducted to obtain the interface properties between shear pins and the compacted material. The apparatus and schematic diagram are illustrated in **Fig. 3-26** and **Fig. 3-27**, respectively. The cuboid container was fixed on a table. Its internal space was 14.3 cm in length, 9.7 cm in width and 10 cm in height. The internal wall was pasted with Teflon sheets to decrease the side friction. Two holes were opened on two sides for the insertion of the shear pin. A cover was used to transfer the above weight into uniform vertical pressure (OUCH et al., 2015).

The pull-out force was provided by a string passing a fixed pulley. The string was tied with the shear pin on the one head and attached with a dangling plastic container on the other head. A target made by Teflon was pasted on the head of the shear pin for the laser sensor. High precision pressure sensor type SSKP325S-02, having a capacity of 19.6 kPa and a radius of 12 mm, was employed in testing (see **Fig. 3-28 (a)**). The adopted laser sensor from Keyence type LK-G400A had a measurement distance of ± 100 mm (see **Fig. 3-28 (b)**).

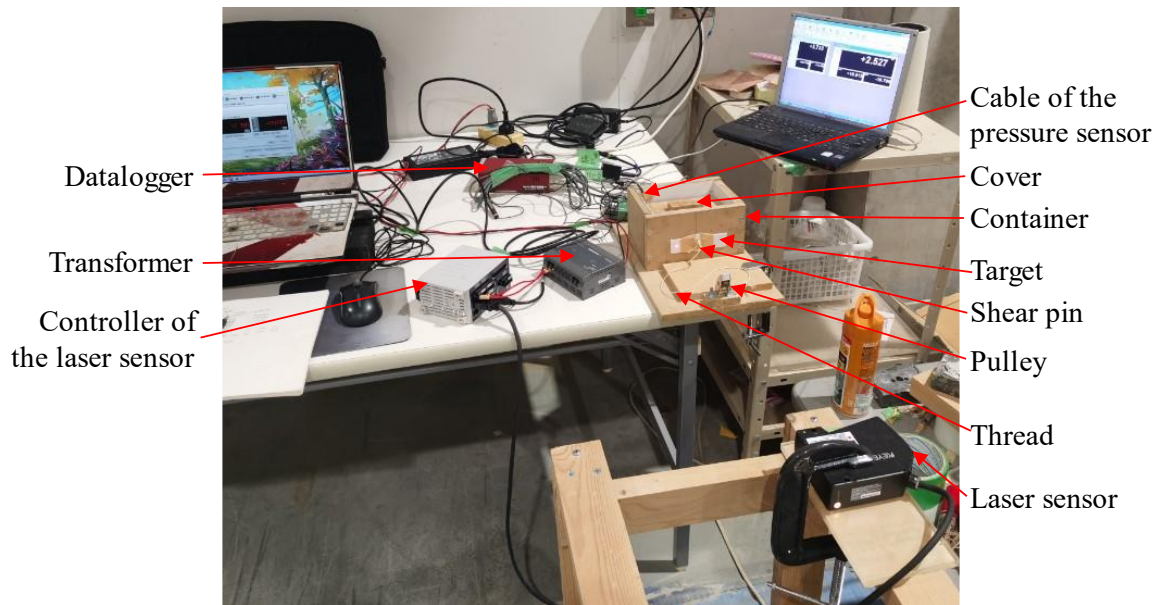


Fig. 3-26 Apparatus of the pull-out test

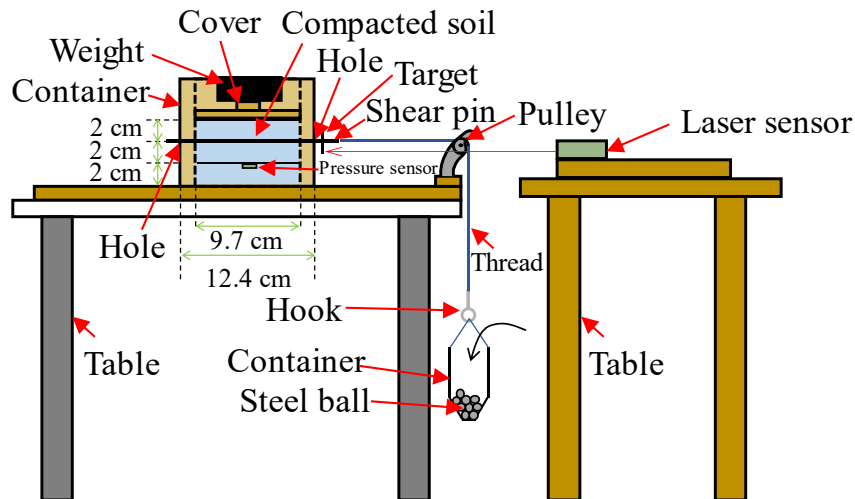


Fig. 3-27 Apparatus' schematic diagram for the pull-out test

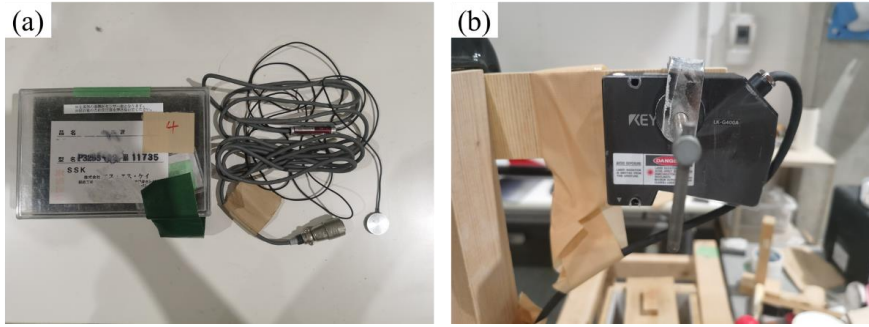


Fig. 3-28 Sensors used in the pull-out test: (a) SSKP325S-02 type pressure sensor; (b) laser sensor from Keyence type LK-G400A

3.4.2 Test procedure

Because the wooden stick and the screw bolt were used as shear pins for the dynamic cases in the centrifugal model tests, where soil with a compaction degree (D_c) of 90% and water content (w) of 8% was adopted, the same pins, D_c and w as the centrifuge test were applied in the pull-out test. The material was filled into the box by three layers sequentially, 2cm for each layer (see **Fig. 3-27**). The earth pressure sensor was initialized to zero and positioned on the upper surface of the first layer, with its recording started onwards. The shear pin with the target was then inserted after the compaction of the second layer. The cover and weight were positioned on the upper surface of the third layer.

The laser sensor was initialized at the beginning of the test. Next, the head of the shear pin and the plastic bottle container were connected by a string and a hook. Subsequently, steel spheres were put into the container bottle gradually until the pin was pulled out or an abrupt jump of the displacement happened. Two steel spheres were added each time, and the accumulated number of them and the stabilized value of displacement of the target were read and recorded by hand. Five different weights were applied for the wooden stick, and six different weights were applied for the screw bolt.

3.4.3 Test results

The actual normal stress (σ_n) on shear pins was obtained through distributing the total weight above the pin by the container's cross-section. The shear stress (τ) was calculated by dividing the total pull-out load, provided by a hanging plastic bottle and the steel balls inside, by the surface area of the pin.

Fig. 3-29 illustrates the correlation between σ_n and the τ required to pull the pins out. By linear regression of the scattering experimental data, two fitting lines are obtained for two pins. The friction angle and the adhesion value between the material and the wooden stick are 33.1° and 0.402 kPa, respectively. The friction angle and the adhesion value between the material and the screw bolt are 41.6° and 0.085 kPa.

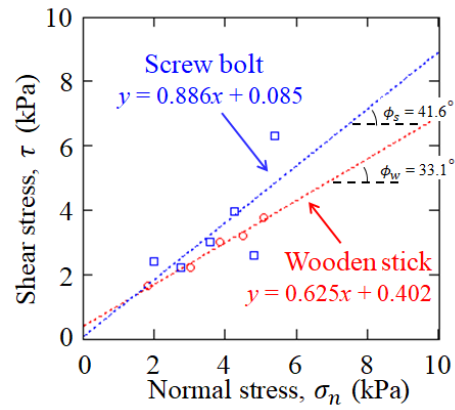


Fig. 3-29 Shear stress vs. normal stress of two kinds of shear pins in pull-out tests

3.5 Bending test

3.5.1 Apparatus of bending test

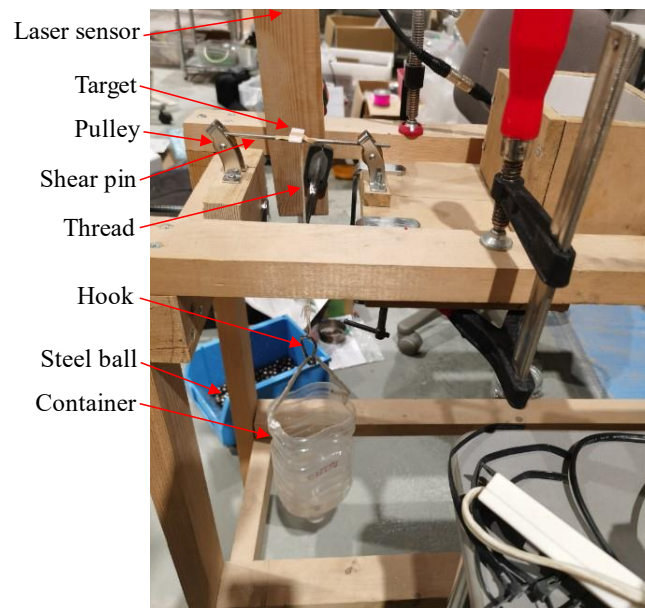


Fig. 3-30 Apparatus of the bending test

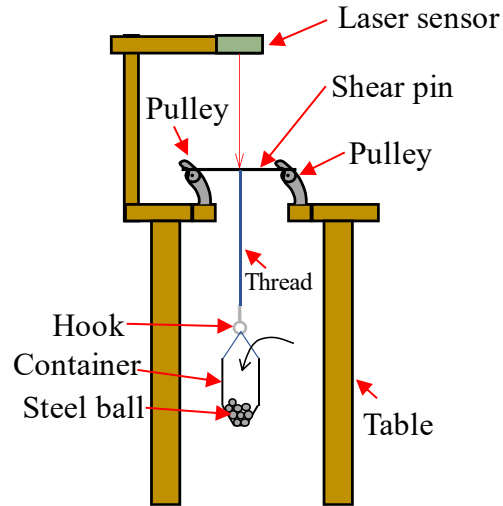


Fig. 3-31 Apparatus' schematic diagram for the bending test

Bending experiments were employed to determine properties of shear pins. As illustrated in **Fig. 3-30** and **Fig. 3-31**, two fixed pulleys and the shear pin were arranged to form a simply supported beam for the test. The distance between the centers of the two pulleys was 16.2 cm for the wooden stick and 12.5 cm for the screw bolt.

A central load was applied using a string that supported a plastic container for steel spheres via a hook, tied at the center of the pin. The hook, container, and each steel sphere had masses of 10.4 g, 30.3 g, and 8.3 g, respectively. A small Teflon target was attached to the center of the pin, allowing deflection measurements to be recorded by an overhead laser sensor (Keyence LK-G400A, as shown in **Fig. 3-28 (b)**).

3.5.2 Test procedure

The laser sensor was initialized at the start, prior to applying any load to the pin. Steel spheres were then gradually added to the plastic container to apply the load. For the wooden stick, one steel sphere was added at a time up to a total of five, while for the screw bolt, five steel spheres were added at a time up to a total of twenty-five. After reaching the maximum load, the spheres were removed in the same increments for the unloading process. Throughout the experiment, the cumulative number of steel balls and the corresponding deflection measurements were recorded.

3.5.3 Test results

The scattering points in **Fig. 3-32** show relationships between the loading weight and the deflection for both the wooden stick and the screw bolt. The deflection of simply supported beam with center load can be calculated by **Eq. (3-7)**, where F is the loading weight, L is the distance between supports (16.2 cm for the wooden stick and 12.5 cm for the screw bolt), E denotes the Young's Modulus and I means the moment of inertia ($2.807 \times 10^{-12} \text{ m}^4$ for the wooden stick and $7.366 \times 10^{-12} \text{ m}^4$ for the screw bolt).

The incremental form of **Eq. (3-7)** is shown as **Eq. (3-8)**, where a is the slope and can be expressed

by **Eq. (3-9)**, and b is the intercept and can be seen as an initial error. By linear regression, two dash lines can be drawn with different a and b for the wooden stick and the screw bolt. After obtaining the value of a , corresponding E can be acquired as **Eq. (3-10)**. The measured E of the wooden stick and the screw bolt are 16 GPa and 114 GPa, respectively.

$$\delta = FL^3/48EI \quad (3-7)$$

$$\delta = aF + b \quad (3-8)$$

$$a = L^3/48EI \quad (3-9)$$

$$E = L^3/48la \quad (3-10)$$

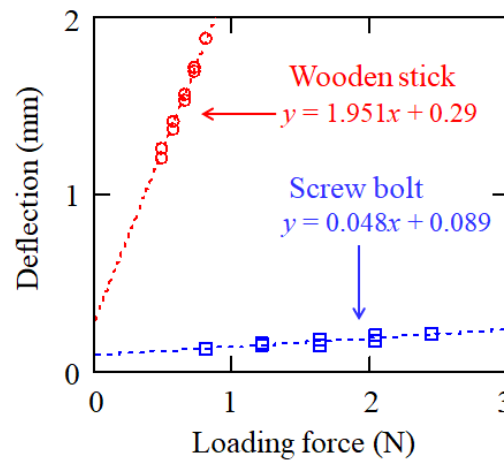


Fig. 3-32 Deflection versus loading force for the bending test

Here, properties of interface between shear pins and compacted Hiroshima sand ($D_c=90\%$ and $w=8\%$) and their bending stiffness, tested in **Section 3.4** and **Section 3.5**, are listed as **Table 3-11**.

Table 3-11 Two types of shear pins' properties

Type	Friction angle (ϕ)	Adhesion (c)	Bending stiffness (EI)
Wooden stick	33.1°	0.402 kPa	0.045 Nm ²
Screw bolt	41.6°	0.085 kPa	0.840 Nm ²

3.6 Soil moisture sensor calibration

3.6.1 Apparatus for the calibration

Calibration of the soil moisture sensor was carried out to determine the correlation between output voltage and soil's water content. In addition to equipment shown in **Fig. 3-5**, a plastic beaker with a maximum capacity of 1,000 ml was selected as the container for the calibration process (see **Fig. 3-33**). The diagram of the calibration system is schematized in **Fig. 3-34**.



Fig. 3-33 Container for the calibration process

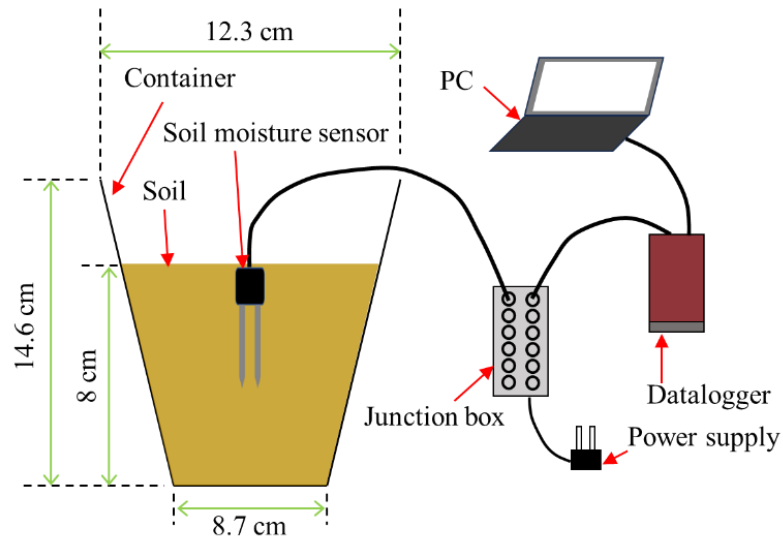


Fig. 3-34 Apparatus' schematic diagram for calibration of a soil moisture sensor

3.6.2 Test procedure

The calibration of soil moisture sensors was carried out separately with $D_c=80\%$ and $D_c=90\%$. A volume of 600 cm^3 was used for calibration. Firstly, the dry sand was filled into the beaker, having the weight determined based on the volume and the dry unit weight in different D_c . The surface of the sand was scraped evenly, and the head of the moisture sensor was then fully inserted into the sand at the center of the sample. The voltage reading displayed on the laptop screen was recorded.

Subsequently, the sensor head was taken out, and water was added to the sand. The sand was then blended uniformly with the water to obtain the required water content of the next measurement. It was compacted using a hammer to maintain the 600 cm^3 volume. Next, the sensor head was installed, and the voltage reading was recorded again. This procedure was repeated until the output voltages for all

required water content levels were measured.

For $D_c=80\%$, measurements were conducted for seven water content levels ($w=0\%$, 2%, 4%, 6%, 8%, 10% and 12%). For $D_c=90\%$, measurements were conducted for nine water content levels ($w=0\%$, 1%, 2%, 4%, 6%, 8%, 10%, 12% and 14%).

3.6.3 Test results

The measured voltages of the moisture sensor with the varying water content are presented as scattering points in **Fig. 3-35**. Hyperbolic function is used to obtain the fitting curve from those points for each D_c . The curve has a larger initial slope when $D_c=90\%$. These calibration curves can be applied in the centrifugal experiments to convert the measured voltage into corresponding water content values.

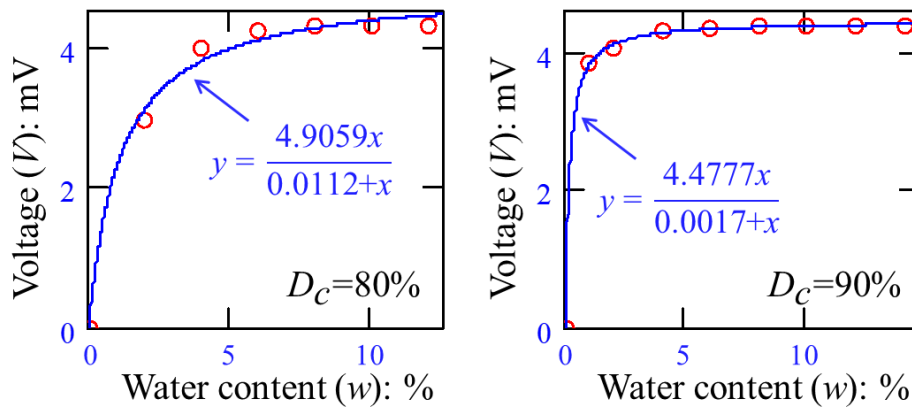


Fig. 3-35 Output voltage of the moisture sensor versus water content

3.7 Summary

This chapter introduces basic laboratory tests that form the foundation for the subsequent research. Direct shear tests (DST) and unconfined compression tests (UCT) were carried out to determine strength parameters for Hiroshima sand. Pull-out and bending experiments were used to evaluate the properties of shear pins for seismic slope retrofitting. The soil moisture sensor was calibrated to monitor water distribution in the slope model during testing. The findings are presented as follows.

(1) Hiroshima sand can be classified between poorly graded and well-graded material. Its properties were significantly influenced by compaction degree and water content. In this study, it was categorized as neither damp or humid sand, depending on the preparation conditions.

(2) In the DST, at a compaction degree of 80%, contraction was observed in dry samples, while dilation occurred in unsaturated samples. The peak strengths were only slightly higher than the residual strengths. Failure envelopes for different water contents (ranging from 6–10%) appeared parallel, differing primarily in their intercepts. This suggests that suction mainly influences apparent cohesion rather than the friction angle in Hiroshima sand under these conditions. When the compaction degree increased to 90%, all samples exhibited dilation. The stress-strain curves for unsaturated samples displayed distinct early peaks, with the average slope of peak strength under unsaturated conditions being steeper than that of residual strength.

(3) The unconfined compressive strengths (UCS) from the UCT were obtained to support the outcomes of the DST. In the stress space, consistency has been found between the UCS and the failure line from DST. For samples with 80% compaction degree, the UCS increased with w up to the optimum water content (w_{opt}). For samples with 90% compaction degree, the UCS increased firstly, then decreases after $w > 8\%$.

(4) Pull-out tests revealed that the interface friction between the screw bolt and Hiroshima sand was higher than that between the wooden stick and the sand, although the adhesion of the wooden stick was slightly higher. Bending tests indicated that the screw bolts' bending stiffness is significantly larger than wooden sticks'.

(5) The calibration curve of the moisture sensor in soil with an 80% compaction degree had a gentler initial slope compared to that in soil with a 90% compaction degree. The output voltage of the moisture transducer increased sharply with rising water content, particularly when the soil is relatively dry.

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Chapter 4 Centrifugal modeling of steep uniform slopes under static loading conditions

In this chapter, the closed-form solution of Culmann's method under 2D static conditions is first derived. This solution provides an explicit solution to a finite uniform slope's factor of safety (FS) without needing the iterative process required by traditional Culmann's method. Additionally, the Culmann's method is extended to the 3D static condition by considering lateral resistance based on the reduction factor (r_d) (see **Section 4.1**).

Subsequently, a series of centrifugal model tests with steep uniform slope models are designed to verify the proposed theories. The experimental scheme, described in **Section 4.2**, includes six cases involving twelve model slopes. These cases examine slopes constructed with Hiroshima sand at two compaction degrees (D_c) under various boundary conditions. The properties of Hiroshima sand have been tested in **Chapter 3**.

In **Section 4.3**, the recorded centrifugal accelerations at the sliding initiation and the observed instability modes of the slope models are described. Additionally, data from installed sensors is presented to illustrate variations in earth pressure and slope displacement during the tests.

In **Section 4.4**, comparison of the experimental results with the closed-form solution of Culmann's method under 2D conditions is presented. The transition from a passive to an active state in the slopes with increasing centrifugal acceleration is also discussed, followed by a 3D theoretical analysis.

Lastly, the conclusions obtained in this chapter are presented in **Section 4.5**.

4.1 Theoretical consideration

4.1.1 Culmann's method under 2D conditions

As schematized in **Fig. 4-1**, Culmann (1866) assumed a planar slip failure or a translational failure of a finite slope where α is the angle of the failure plane, β is the angle of the slope face, and h is the height of the slope. The expressions for the slope face length, s , and the failure plane length, l , are given below.

$$s = h \cdot \csc \beta \quad (4-1)$$

$$l = h \cdot \csc \alpha \quad (4-2)$$

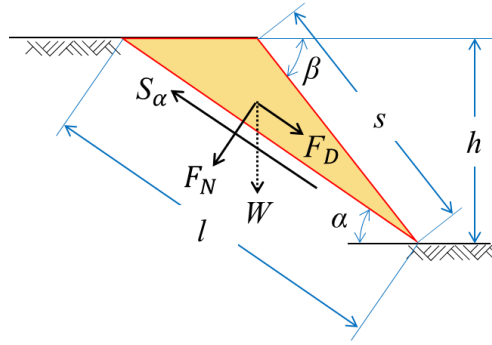


Fig. 4-1 Schematics of Culmann (1866)'s method of planar wedge

Correspondingly, the weight, W , of the finite slope under the acceleration of $n \cdot g$, the normal load, F_N , as well as the driving load, F_D , acting on the failure plane are obtained using **Eq. (4-3)**, **Eq. (4-4)** and **Eq. (4-5)**, respectively. Herein, ρ_t denotes material's total unit weight, n is the scale factor in centrifuge modelling to account for gravity, and g represents the gravitation acceleration, which is typically 9.81 m/s^2 .

$$W = \frac{1}{2} n \rho_t g h^2 \cdot (\cot \alpha - \cot \beta) \quad (4-3)$$

$$F_N = W \cos \alpha \quad (4-4)$$

$$F_D = W \sin \alpha \quad (4-5)$$

Assuming that the shear properties of the ground can be described with the cohesion, c , and the friction angle, ϕ . The shear resisting force along the failure plane, inclined at an slope of α , is governed by S_α expressed below.

$$S_\alpha = cl + F_N \tan \phi \quad (4-6)$$

The factor of safety, FS , is calculated in **Eq. (4-7)**. Therefore, the slope fails when F_D reaches S_α , which occurs as n is increased in the centrifuge model.

$$FS = \frac{S_a}{F_D} \quad (4-7)$$

The following expression is obtained by substituting **Eqs. (4-1) to (4-6)** into **Eq. (4-7)**, where the dimensionless χ , referred to as the stability ratio, is described for simplicity.

$$FS = \frac{4\sin\beta}{\cos(2\alpha-\beta)-\cos\beta}\chi + \frac{\tan\phi}{\tan\alpha} \quad (4-8)$$

$$\text{Where } \chi = \frac{c}{n\rho_t gh} \quad (4-9)$$

The value of FS cannot be directly calculated by **Eq. (4-8)** because the value of α is unknown. With regard to **Eq. (4-8)**, the partial differentiation of FS with α is obtained as follows.

$$\frac{\partial}{\partial\alpha} FS = \frac{8\sin(2\alpha-\beta)\sin\beta}{(1-\cos2(\beta-\alpha))(1-\cos2\alpha)} \chi - \frac{\tan\phi}{\sin^2\alpha} \quad (4-10)$$

Minimization of **Eq. (4-8)** requires $\partial FS/\partial\alpha=0$; therefore, the optimum χ can be solved.

$$\chi = \frac{\tan\phi}{2} \frac{1-\cos2(\beta-\alpha)}{\cos2(\beta-\alpha)-\cos2\alpha} \quad (4-11)$$

Substituting of **Eq. (4-11)** into **Eq. (4-8)** can formulate the minimum FS as follows.

$$FS = \frac{\tan\phi}{\tan(2\alpha-\beta)} \quad (4-12)$$

The variation of FS or **Eq. (4-8)** with the unknown α for a given set of χ , β and ϕ is demonstrated in **Fig. 4-2** together with the trace of the minimum FS or **Eq. (4-12)**.

As a consequence of minimization, **Eq. (4-12)** results in the following equations. The developed friction angle, ϕ_d , and the developed friction coefficient, $\tan(\phi_d)$, are defined, respectively.

$$\phi_d = 2\alpha - \beta \quad (4-13)$$

$$\tan\phi_d = \frac{\tan\phi}{FS} \quad (4-14)$$

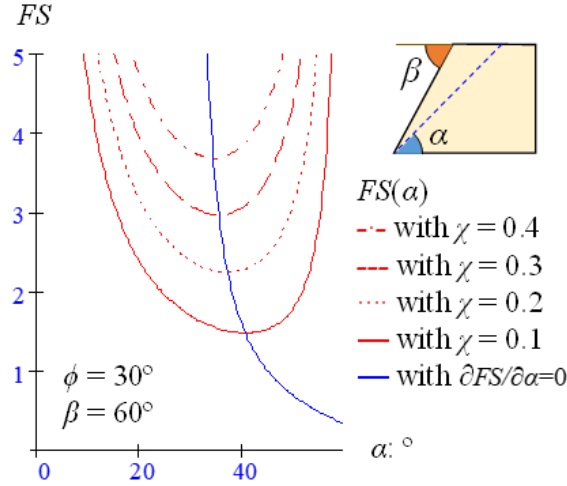


Fig. 4-2 Variation of the FS and its minimum with respect to the failure plane angle

From **Eq. (4-13)**, the corresponding α for achieving the minimum FS can be solved.

$$\alpha = \frac{\beta + \phi_d}{2} \quad (4-15)$$

The corresponding χ for achieving the minimum FS can be obtained by substitution of **Eq. (4-15)** into **Eq. (4-11)**.

$$\chi = \frac{1 - \cos(\beta - \phi_d)}{4 \sin \beta \sin \phi_d} \tan \phi \quad (4-16)$$

Therefore, the required n is determined from **Eq. (4-16)**, using **Eq. (4-9)** and **Eq. (4-14)**. The developed cohesion c_d is defined by **Eq. (4-18)**.

$$n = \frac{4c_d}{\rho_t g h} \frac{\sin \beta \cos \phi_d}{1 - \cos(\beta - \phi_d)} \quad (4-17)$$

$$\text{Where } c_d = \frac{c}{FS} \quad (4-18)$$

Hence, **Eq. (4-14)**, **Eq. (4-18)**, and **Eq. (4-7)** implies the concept of factor of safety for an arbitrary value of F_N acting on the failure surface, as introduced in many literatures, e.g., Bishop (1960) and Janbu (1973).

$$FS = \frac{cL + F_N \tan \phi}{c_d L + F_N \tan \phi_d} = \frac{c}{c_d} = \frac{\tan \phi}{\tan \phi_d} \quad (4-19)$$

Also, the required β for a given n is determined by expanding **Eq. (4-17)** and rearranging it to the following equation where $\tan\zeta$ is introduced for simplicity.

$$\cos\beta + \tan\zeta\sin\beta = \frac{1}{\cos\phi_d} \quad (4-20)$$

$$\text{Where } \zeta = \tan^{-1}\left(\tan\phi_d + \frac{4c_d}{n\rho_t g h}\right) \quad (4-21)$$

Therefore, **Eq. (4-20)** simply leads to the solution of β given below.

$$\beta = \zeta + \cos^{-1}\left(\frac{\cos\zeta}{\cos\phi_d}\right) \quad (4-22)$$

When $FS = 1$, **Eq. (4-15)**, **Eq. (4-17)** and **Eq. (4-22)** lead to the failure plane angle, α_f , the scale factor, n_f , and slope angle at failure, β_f , respectively.

$$\alpha_f = \frac{\beta + \phi}{2} \quad (4-23)$$

$$n_f = \frac{4c}{\rho_t g h} \frac{\sin\beta \cos\phi}{1 - \cos(\beta - \phi)} \quad (4-24)$$

$$\beta_f = \tan^{-1}(\tan\phi + 4\chi) + \cos^{-1}\left(\frac{1}{\sqrt{1 + (\tan\phi + 4\chi)^2 \cos\phi}}\right) \quad (4-25)$$

Accordingly, the critical height of slope, h_f , is determined from **Eq. (4-24)** as follows.

$$h_f = \frac{4c}{n\rho_t g} \frac{\sin\beta \cos\phi}{1 - \cos(\beta - \phi)} \quad (4-26)$$

The height of unsupported vertical cut, h_o , can be obtained by substituting $\beta = 90^\circ$ into **Eq. (4-26)**.

$$h_o = \frac{4c}{n\rho_t g} \frac{\cos\phi}{1 - \sin\phi} \quad (4-27)$$

The dimensionless slope face length, s/h_o , and the dimensionless failure plane length, l/h_o , are obtained from **Eq. (4-1)** and **Eq. (4-2)**, respectively.

$$\frac{s}{h_o} = \frac{h_f}{h_o} \csc\beta \quad (4-28)$$

$$\frac{l}{h_o} = \frac{h_f}{h_o} \csc \alpha \quad (4-29)$$

Form **Eq. (4-26)** and **Eq. (4-27)**, the ratio of h_f to h_o is expressed as **Eq. (4-30)**.

$$\frac{h_f}{h_o} = \frac{1 - \sin \phi}{1 - \cos(\beta - \phi)} \sin \beta \quad (4-30)$$

Following Jaky (1936), the limiting curve for the slope face based on **Eq. (4-28)** and a given friction angle is demonstrated in **Fig. 4-3** using polar coordinates. This graphically explains that the limiting slope height can be increased if the slope face angle is sufficiently decreased. Additionally, the limiting curve for the failure plane based on **Eq. (4-29)** is included to delineate the outline of the planar wedge, illustrated by h_f/h_o for 3 different values.

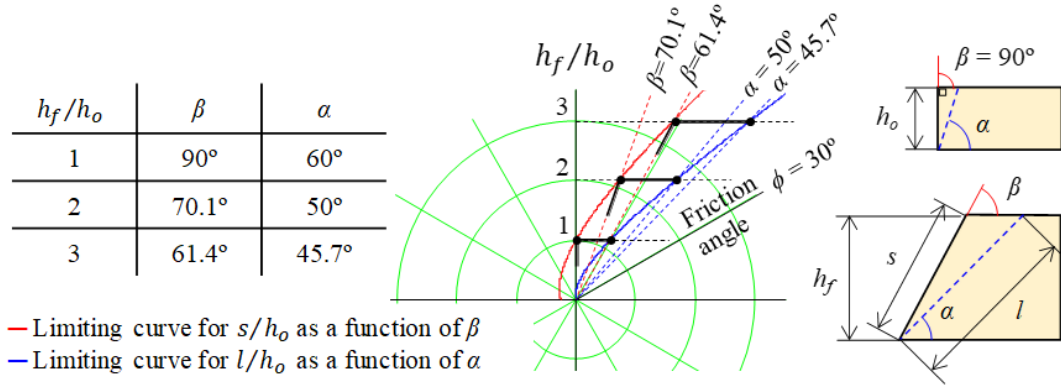


Fig. 4-3 Limiting curves in polar coordinates for the slope face (dimensionless length s/h_o vs angle β) and the failure plane (dimensionless length l/h_o vs angle α)

The alternative approach can be considered by averaging the normal stress, σ_α , and the shear stress, τ_α , acting on the failure surface inclined at an inclination of α , and then finding the developed shearing resistance, τ_d , as follows.

$$\sigma_\alpha = \frac{F_N}{l} = \frac{1}{2} n \rho_t g h \frac{\sin(\beta - \alpha) \cos \alpha}{\sin \beta} \quad (4-31)$$

$$\tau_\alpha = \frac{F_D}{l} = \frac{1}{2} n \rho_t g h \frac{\sin(\beta - \alpha) \sin \alpha}{\sin \beta} \quad (4-32)$$

$$\tau_d = c_d + \sigma_\alpha \tan \phi_d \quad (4-33)$$

The expression of c_d can be formulated below by equating $\tau_\alpha = \tau_d$ for an arbitrary plane.

$$c_d = \frac{1}{2} n \rho_t g h \frac{\sin(\beta - \alpha) \sin(\alpha - \phi_d)}{\sin \beta \cos \phi_d} \quad (4-34)$$

The angle α that gives the maximum c_d is obtained from $\partial c_d / \partial \alpha = 0$, leading to the same equation derived earlier in **Eq. (4-15)**. At failure when $c_d = c$ and $\phi_d = \phi$, the same solutions as given in **Eq. (4-23)** to **Eq. (4-26)** are achieved. Substitution of **Eq. (4-23)** into σ_α and τ_α yields the average normal stress, σ_f , with the shear stress, τ_f , at the failure surface.

$$\sigma_f = \frac{1}{2} n \rho_t g h \frac{\sin \beta - \sin \phi}{2 \sin \beta} \quad (4-35)$$

$$\tau_f = \frac{1}{2} n \rho_t g h \frac{\cos \phi - \cos \beta}{2 \sin \beta} \quad (4-36)$$

The above equations satisfy the Mohr-Coulomb failure standard, which can be expressed below when $n = n_f$, as described in **Eq. (4-24)**.

$$\tau_f = c + \sigma_f \tan \phi \quad (4-37)$$

For a given condition of slope, FS is evaluated through iterative calculations by assuming an initial guess for FS with a value greater than one to compute ϕ_d in **Eq. (4-14)**, updating α in **Eq. (4-15)**, and recalculating FS in **Eq. (4-8)** until the convergence to a stable value of FS . To avoid the tedious iterative procedure, the derivation of the closed-form of FS is newly proposed hereafter.

4.1.2 Closed-form solution for Culmann's method under 2D conditions

By replacing β with **Eq. (4-22)** and ζ with **Eq. (4-21)**, the expansion of **Eq. (4-20)** can be rearranged to **Eq. (4-38)**, which leads to the solution of FS derived in **Eq. (4-39)** by substitution χ with **Eq. (4-9)** and selecting the root for $FS > 0$. Henceforth, the FS for a uniform slope under 2D conditions can be calculated directly, eliminating the need for a tedious iterative process.

$$\cos \beta + \frac{4\chi + \tan \phi}{FS} \sin \beta = \sqrt{1 + \left(\frac{\tan \phi}{FS} \right)^2} \quad (4-38)$$

$$FS = \frac{(\tan \phi + 4\chi) \cos \beta + 2\sqrt{(\tan \phi + 2\chi)2\chi}}{\sin \beta} \quad (4-39)$$

The variation of FS with ϕ for a given set of χ and β and the variation of FS with β for a given set of χ and ϕ are illustrated by plotting **Eq. (4-39)** in **Fig. 4-4**.

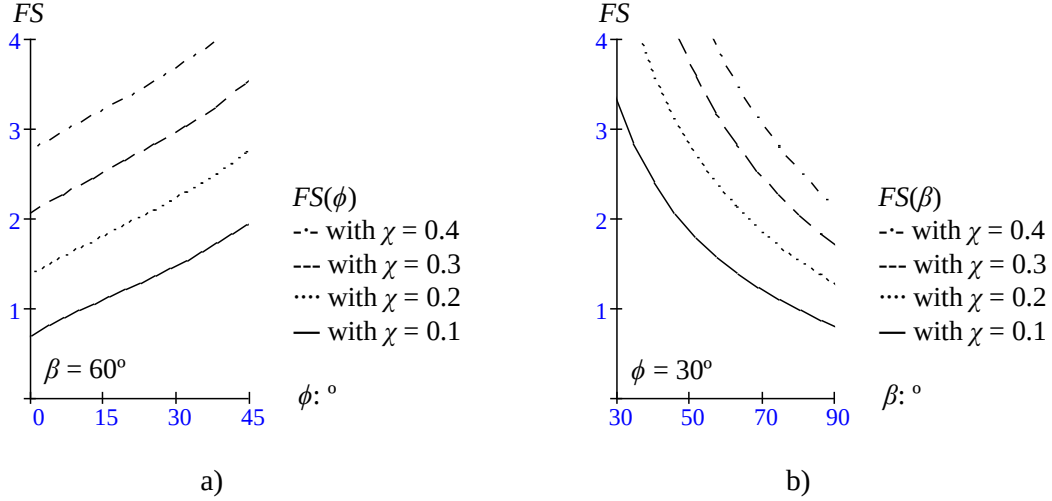


Fig. 4-4 Variations of FS with the friction angle and the slope inclination: (a) variation of FS with the friction angle for the slope inclination of 60° ; (b) variation of FS with the slope inclination for a friction angle of 30°

4.1.3 Extension of Culmann's method to 3D condition

According to Rankine's theory, horizontal effective stress changes with the vertical effective stress. The expressions of passive and active earth pressure are as follows, represented by σ'_{ph} and σ'_{ah} , respectively.

$$\sigma'_{ph} = \tan^2 \left(\frac{\pi}{4} - \frac{\phi}{2} \right) \sigma'_v - 2c \tan \left(\frac{\pi}{4} - \frac{\phi}{2} \right) \quad (4-40)$$

$$\sigma'_{ah} = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \sigma'_v + 2c \tan \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \quad (4-41)$$

$$\text{Where } \sigma'_v = \rho_c g h \quad (4-42)$$

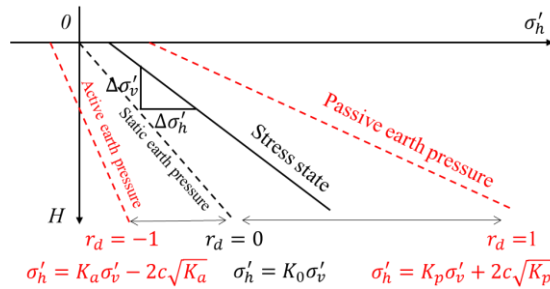


Fig. 4-5 Change of state reflected by r_d

Concerning shear strength reduction technique (SSR) (Dawson et al., 1999; Pipatpongsa et al., 2022), the intermediate state between passive and active can be expressed by using shear strength reduction ratio r_d (shown in **Fig. 4-5**) and coefficient of lateral earth pressure K as Eq. (4-43), (4-44) and (4-45).

The variation of K from the active coefficient, K_a , to the passive coefficient, K_p , can be related with the range of r_d from -1 to 1. As r_d can be negative in an active state, this study refers to r_d as the reduction ratio in order to avoid confusion with the shear strength reduction ratio that is positively defined for the shear strength reduction technique (Pipatpongsa et al., 2024).

$$\sigma'_h = K\sigma'_v + 2r_d c\sqrt{K} \quad (4-43)$$

$$\sqrt{K} = \tan\left(\frac{\pi}{4} + \frac{\tan^{-1}(r_d \tan\phi)}{2}\right) \quad (4-44)$$

$$\text{Where } r_d = \frac{\sigma'_h - \sigma'_v}{2\sqrt{(\sigma'_v \tan\phi + c)(\sigma'_h \tan\phi + c)}} \quad (4-45)$$

Taking the developed lateral pressure into consideration by adapting the concept of the SSR, the 2D analysis can be expanded to 3D situation. To calculate the lateral normal force on one side of the sliding block, the side's shape can be equivalent to the triangle shown in right side of (see **Fig. 4-6**).

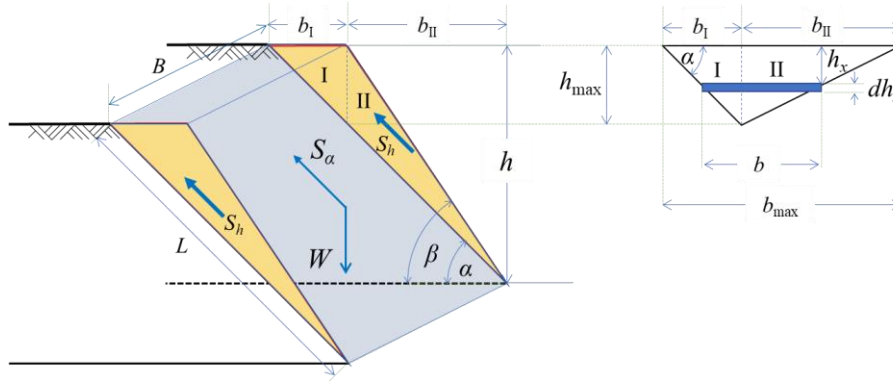


Fig. 4-6 Sliding block's force diagram under 3D conditions

In the equivalent triangle,

$$b_I = h(\cot\alpha - \cot\beta) \quad (4-46)$$

$$b_{II} = h \cot\beta \quad (4-47)$$

$$b_{max} = b_I + b_{II} \quad (4-48)$$

$$h_{max} = b_I \tan\alpha \quad (4-49)$$

The width of the small strip in the equivalent triangle is expressed by Eq. (4-50). Its area and the total lateral force on one side are obtained as Eq. (4-51) and (4-52), respectively. After integration, total normal force on one side is expressed as Eq. (4-53).

$$b = (1 - \frac{h_x}{h_{max}}) b_{max} \quad (4-50)$$

$$dA_h = b dh_x \quad (4-51)$$

$$F_h = \int_0^{h_{max}} \sigma'_h dA_h \quad (4-52)$$

$$F_h = \frac{1}{6} K \rho_g n h c \cot \alpha [h(1 - \cot \beta \tan \alpha)]^2 + r_d c \sqrt{K} h c \cot \alpha [h(1 - \cot \beta \tan \alpha)] \quad (4-53)$$

The resisting force along one side is calculated through the Mohr-Coulomb failure criterion, as shown in Eq. (4-54). ϕ_i is the interface friction angle and c_i is the adhesion between the material and the wall. The FS can be obtained as Eq. (4-55) and (4-56), taking into account the resisting force on both sides of the block. The lateral resistance provided by side walls can be seen as additional cohesion, represented by Eq. (4-57).

$$S_h = F_h \tan \phi_i + A_h c_i \quad (4-54)$$

$$FS = \frac{S_a + 2S_h}{F_D} \quad (4-55)$$

$$FS = \frac{\tan \phi}{\tan \alpha} + (1 + 2 \frac{h}{B} \frac{(2r_d + \sqrt{K} \frac{1}{3} (\frac{\cos(2\alpha - \beta) - \cos \beta}{\sin 2\alpha \sin \beta} (\frac{n \rho_g h}{c}))) \sqrt{K} \tan \phi_i + \frac{c_i}{c}}{4 \sin \alpha \sin \beta \cos(2\alpha - \beta) - \cos \beta}) \frac{4 \sin \beta}{\cos(2\alpha - \beta) - \cos \beta} \frac{c}{n \rho_g h} \quad (4-56)$$

$$\Delta c = 2 \frac{h}{B} \frac{(2r_d + \sqrt{K} \frac{1}{3} (\frac{\cos(2\alpha - \beta) - \cos \beta}{\sin 2\alpha \sin \beta} (\frac{n \rho_g h}{c}))) \sqrt{K} \tan \phi_i + \frac{c_i}{c}}{4 \sin \alpha \sin \beta \cos(2\alpha - \beta) - \cos \beta} \quad (4-57)$$

The critical angle α can also be obtained by differentiation theoretically. Due to the complexity of FS in 3D condition, it is difficult to get an explicit expression of α , but it can be calculated in a numerical way.

4.2 Experimental scheme

4.2.1 Program of the test

Centrifugal model tests were carried out to figure out the failure mechanism of steep uniform slopes under static loading. The centrifuge facility utilized for these tests is located at the Geotechnical Centrifuge Center, Disaster Prevention Research Institute (DPRI), Kyoto University, featuring an effective radius of 2.5 m, maximum payload of 24 G×ton and can achieve a maximum centrifugal acceleration of 200G (Tao & Pipatpongsa, 2022).

In this part, six static centrifugal models were tested, including twelve model slopes. For each case, two model slopes were tested simultaneously to increase efficiency and to facilitate the comparison of the effects of different slope angles. **Fig. 4-7** illustrates built slope models before loading, with all model

slopes having the same total height and the slope height, which are 18 cm and 15 cm at model scale, respectively. Case 1 to Case 3 was built with humid Hiroshima sand under $D_c=80\%$, and Case 4 to Case 6 was built with damp Hiroshima sand under $D_c=90\%$. For all cases, $w=8\%$. In particular, the boundary gaps of 0.5 cm in case 6 were created by placing Teflon sheets on the glass side and back wall before compaction and carefully pulling them out after the slope cut.

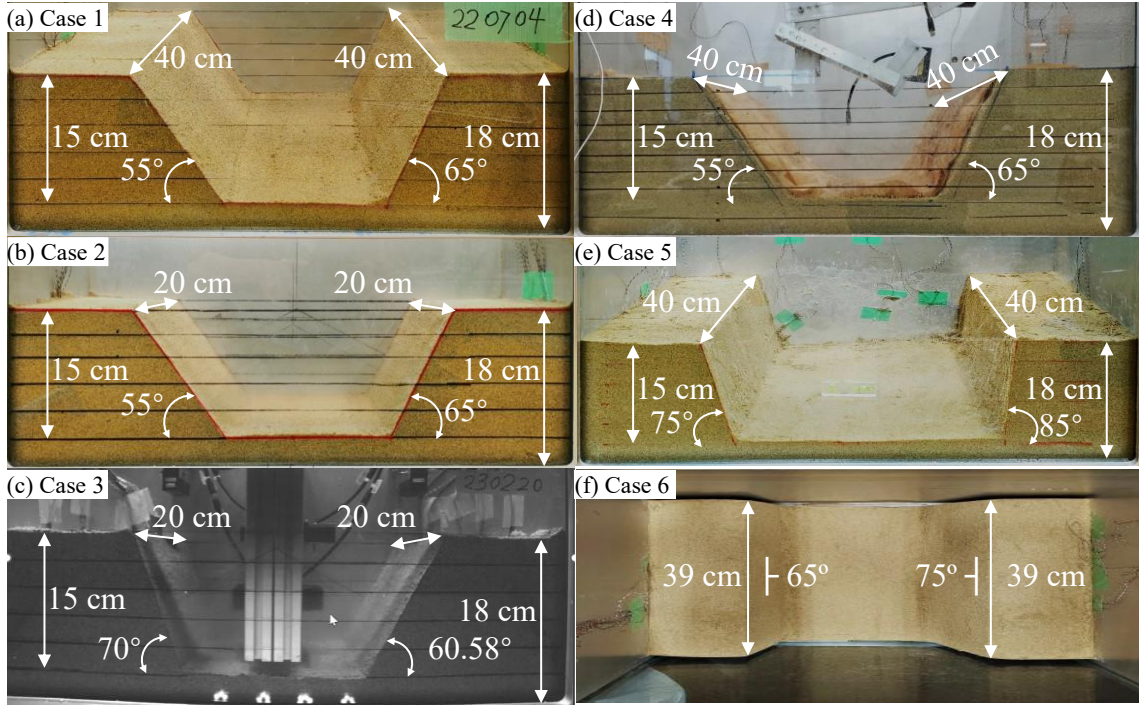


Fig. 4-7 Established model slopes before loading (model scale)

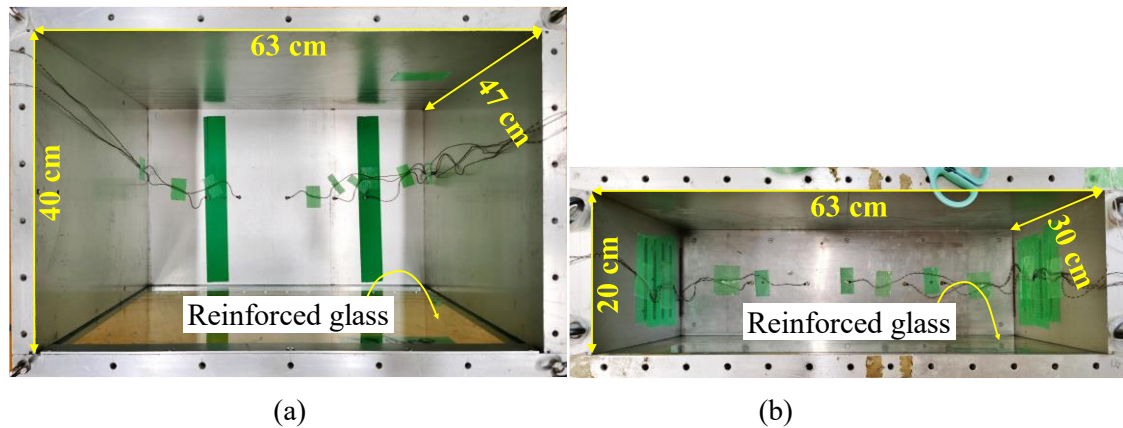


Fig. 4-8 Dimensions of soil chamber in different sizes (model scale): (a) wide soil chamber; (b) narrow soil chamber

Two types of soil chambers with different widths were adopted (see **Fig. 4-8**) to perform distinct boundary conditions. The dimensions of the wide and the narrow soil chamber were 63 cm $\times$ 40 cm $\times$

47 cm, and 63 cm $\times$ 20 cm $\times$ 30 cm (length $\times$ width $\times$ height), respectively, all at model scale. The soil chambers were made of aluminum alloy fabricated with a transparent reinforced glass at the front for visual observation (Tao & Pipatpongsa, 2022).

Table 4-1 shows the detailed model configurations for all three cases, including the case number, experimental date, type of soil chamber used, degree of compaction (D_c), slope angle (β), initial water content (w), and the loading process. A maximum centrifugal loading of 100G was employed in the tests to trigger slope sliding. A diagram illustrating a model slope under centrifugal loading is shown in **Fig. 4-9**.

Table 4-1 Model configurations for steep slopes under static loading

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β ($^\circ$)		Initial water content, w (%)	Loading process (G)
				Left	Right		
1	220704	Wide	80	55	65	8	1 $\rightarrow$ 50 $\rightarrow$ 60 $\rightarrow$ 70 $\rightarrow$ 80 $\rightarrow$ 1
2	220513	Narrow		55	65		1 $\rightarrow$ 50 $\rightarrow$ 60 $\rightarrow$ 70 $\rightarrow$ 80 $\rightarrow$ 1
3	230220	Narrow		70	60.58		1 $\rightarrow$ 50 $\rightarrow$ 1
4	210805	Wide	90	55	65		1 $\rightarrow$ 30 $\rightarrow$ 40 $\rightarrow$ 50 $\rightarrow$ 60 $\rightarrow$ 70 $\rightarrow$ 1
							1 $\rightarrow$ 70 $\rightarrow$ 80 $\rightarrow$ 90 $\rightarrow$ 100 $\rightarrow$ 1
5	210921	Wide		75	85		1 $\rightarrow$ 70 $\rightarrow$ 80 $\rightarrow$ 1
							1 $\rightarrow$ 100 $\rightarrow$ 1
6	211007	Wide		75	65		1 $\rightarrow$ 100 $\rightarrow$ 1

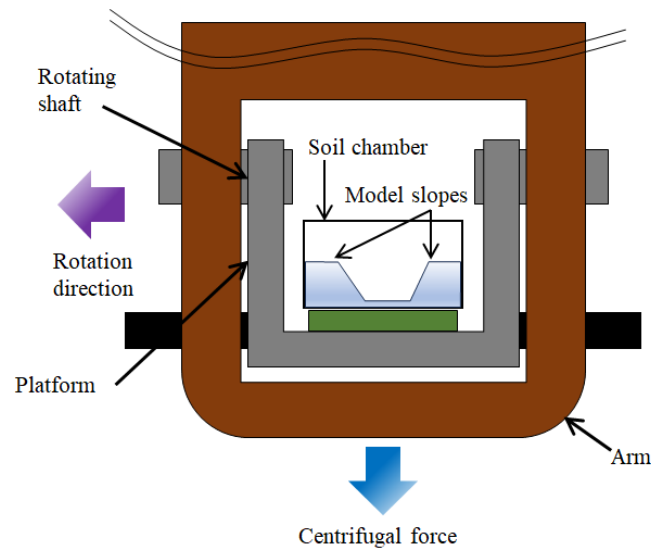


Fig. 4-9 Diagram of model slopes during centrifugal loading

4.2.2 Instrumentation

A maximum of 23 earth pressure gauges, type PDA-500KPB (3.8 mm in radius by 2 mm in height) with a maximum range of 500 kPa, were placed inside models to measure internal earth pressure (see **Fig.**

4-10 (a)). Additionally, up to six laser transducers were installed to monitor the deformation of the slope at specific locations during the test. These sensors were either type IL-065, with a reference distance of 65 mm and a measurable distance of 55 to 105 mm, or type IL-100 with a reference distance of 100 mm and a measurable distance of 75 to 130 mm (see **Fig. 4-10 (b)**).

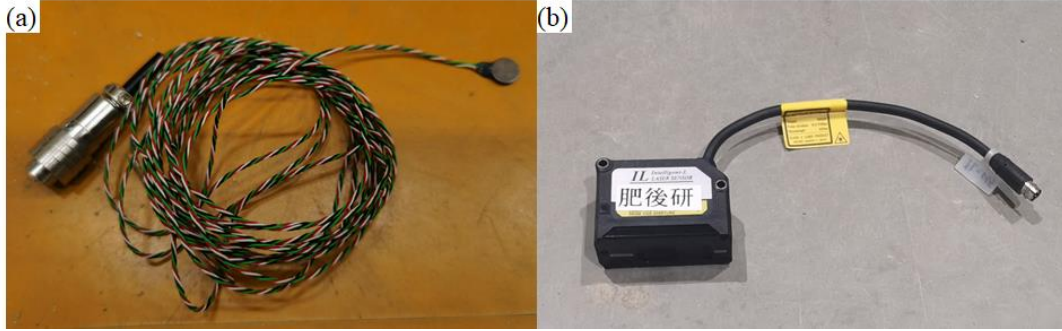


Fig. 4-10 Gauges in the experiments: (a) earth pressure gauge; (b) laser sensor

Furthermore, some earth pressure gauges were consolidated on rigid triangular bases, forming three-directional earth pressure gauges (see **Fig. 4-11**).

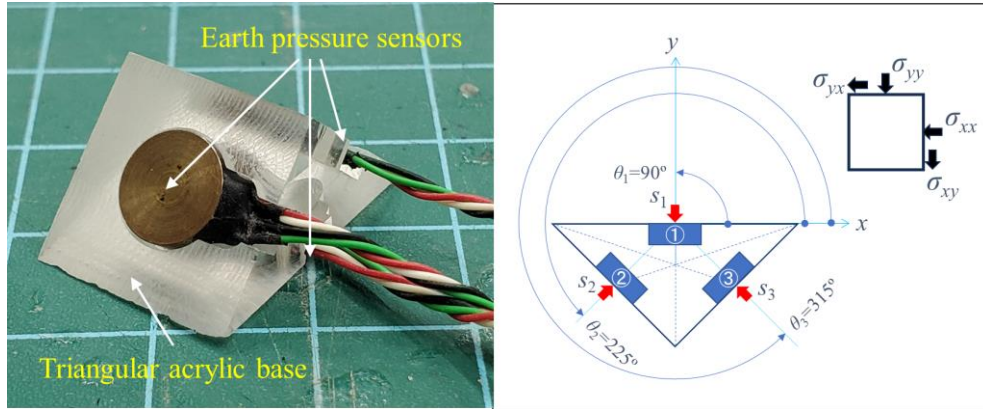


Fig. 4-11 Three-directional earth pressure sensor

This configuration allows for the simultaneous measurement of normal (σ_{xx} and σ_{yy}) and shear stress (σ_{xy}) at a single location for a 2D stress system. The calculation method, based on three values obtained by earth pressure sensors (s_1 , s_2 and s_3), is presented as follows.

Given the stress tensor (σ) for an infinitesimal element in 2D cartesian system in **Eq. (4-58)** and outer normal vector for an arbitrary oblique plane ($\mathbf{n}(\theta)$) in **Eq. (4-59)**, where θ is the rotation from the x -axis to the normal vector. The normal stress' magnitude on that plane can be obtained through **Eq. (4-60)**. Here, the sharing stress equality theorem is taken for granted.

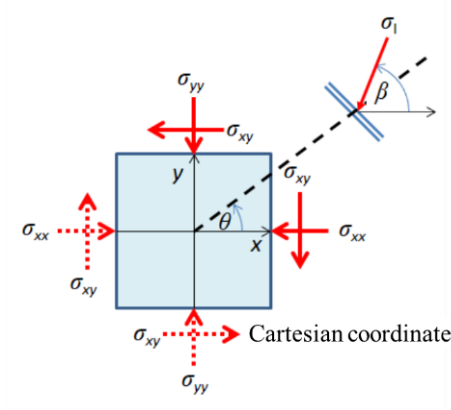


Fig. 4-12 Stress state for an infinitesimal element in 2D coordinate system

$$\boldsymbol{\sigma} = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} \\ \sigma_{yx} & \sigma_{yy} \end{bmatrix} \quad (4-58)$$

$$\mathbf{n}(\theta) = \begin{bmatrix} \cos(\theta) \\ \sin(\theta) \end{bmatrix} \quad (4-59)$$

$$\begin{aligned} \sigma_n &= \{\boldsymbol{\sigma} \cdot \mathbf{n}(\theta)\} \cdot \mathbf{n}(\theta) \\ &= \sigma_{xx} \frac{1 + \cos(2\theta)}{2} + \sigma_{yy} \frac{1 - \cos(2\theta)}{2} + \sigma_{xy} \sin(2\theta) \end{aligned} \quad (4-60)$$

A stress vector ($\mathbf{v}$), which includes three stress components, is shown in **Eq. (4-61)**. The vector ($\mathbf{s}$), which includes the magnitudes of the measured normal stresses in three different planes, is shown in **Eq. (4-62)**. An equation between $\mathbf{v}$ and $\mathbf{s}$ them is derived as **Eq. (4-63)** by **Eq. (4-60)**, where $\mathbf{M}(\theta_1, \theta_2, \theta_3)$ is a coefficient matrix as shown in **Eq. (4-64)**. The $\mathbf{v}$ can be solved by **Eq. (4-65)**, where $\mathbf{M}^{-1}(\theta_1, \theta_2, \theta_3)$ is the inverse matrix of the coefficient matrix.

$$\mathbf{v} = \begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{xy} \end{bmatrix} \quad (4-61)$$

$$\mathbf{s} = \begin{bmatrix} s_1 \\ s_2 \\ s_3 \end{bmatrix} \quad (4-62)$$

$$\mathbf{M}(\theta_1, \theta_2, \theta_3) \cdot \mathbf{v} = \mathbf{s} \quad (4-63)$$

$$\mathbf{M}(\theta_1, \theta_2, \theta_3) = \begin{bmatrix} \frac{\cos(2\theta_1)}{2} + \frac{1}{2} & -\frac{\cos(2\theta_1)}{2} + \frac{1}{2} & \sin(2\theta_1) \\ \frac{\cos(2\theta_2)}{2} + \frac{1}{2} & -\frac{\cos(2\theta_2)}{2} + \frac{1}{2} & \sin(2\theta_2) \\ \frac{\cos(2\theta_3)}{2} + \frac{1}{2} & -\frac{\cos(2\theta_3)}{2} + \frac{1}{2} & \sin(2\theta_3) \end{bmatrix} \quad (4-64)$$

$$\mathbf{v} = \mathbf{M}^{-1}(\theta_1, \theta_2, \theta_3) \cdot \mathbf{s} \quad (4-65)$$

In this study, $\theta_1, \theta_2, \theta_3$ are set to 90° , 225° and 315° , respectively, as illustrated in **Fig. 4-11**. Thus, expressions of three stress components are given as **Eq. (4-66)**. **Eq. (4-67)** shows the rotation from the

x -axis to the major stress.

$$\begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{xy} \end{bmatrix} = \begin{bmatrix} -s_1 + s_2 + s_3 \\ s_1 \\ (s_2 - s_3)/2 \end{bmatrix} \quad (4-66)$$

$$\beta = \frac{1}{2} \text{atan}\left(\frac{2\sigma_{xy}}{\sigma_{xx} - \sigma_{yy}}\right) \quad (4-67)$$

The arrangement of sensors varied slightly in each case. In the first three cases, where $D_c=80\%$, using wide and narrow soil chambers, most of the sensors were placed at the bottom, the one-third height and the two-thirds height of the model (see **Fig. 4-14**, **Fig. 4-15** and **Fig. 4-16**). Eight earth pressure sensors (P6, P7, P8, P9, P12, P13, P14, P15) were placed on the slope boundary to monitor the horizontal earth pressures and nine other earth pressure sensors (P1, P2, P3, P4, P5, P10, P11, P16, P17) were used for the vertical earth pressures. In other three cases, where $D_c=90\%$, using a wide soil chamber, earth pressure transducers were positioned at the bottom and half model height (see **Fig. 4-17**, **Fig. 4-18** and **Fig. 4-19**). Four earth pressure sensors (P6, P7, P8, P9) were placed on the slope boundary to monitor the horizontal earth pressures and other seven earth pressure transducers (P1, P2, P3, P4, P5, P10, P11) have been used for vertical earth pressures. One accelerometer was also put on the crest of the right slope of Case 4 and Case 5 to compare acceleration with the value reported by the centrifuge.

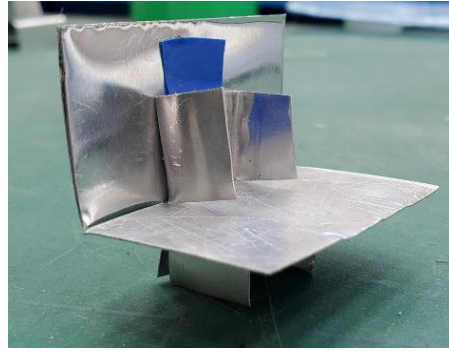


Fig. 4-13 Target of the laser sensor made by steel plate

The displacement of model slopes was also recorded by laser sensors. In Case 1, two laser sensors (LD1 and LD4) were employed to measure the slope crests' vertical displacement on both sides. In Case 2, two additional laser sensors (LD2 and LD3) were deployed to measure the slope crests' horizontal displacement. In Case 3, another two laser sensors (LD3 and LD4) were installed to measure the displacement in the horizontal direction at the middle height of slope surfaces. For all cases with $D_c=90\%$ (Case 4 to Case 6), only two laser sensors (LD3 and LD4) were deployed to monitor the right slope crest's horizontal and vertical displacement. Targets made by steel plate (see **Fig. 4-13**) were inserted on slope crests as targets for laser sensors.

In Case 1 and Case 3, sensors P18, P19, P20, P21, P22 and P23 were combined to create two three-directional earth pressure gauges. These gauges were placed at two-thirds of the model height and positioned between the longitudinal centerline and the back wall of the soil chamber. In Case 5 and Case 6, sensors P12, P13, P14 were combined into one three-directional earth pressure gauge. The gauge was placed at middle height of the model on the longitudinal centerline near the left wall of the soil chamber.

Additionally, in case 3, six water moisture sensors, type GTI-S50 and made by GRI (Geo-research institute), were used to measure the variation of water content in the slopes, placed on the bottom, middle and top of the models (see **Fig. 4-16**). Moreover, high-speed cameras were also deployed in every case for recording slope deformation, providing visual evidence to complement the sensor measurements.

The configuration of sensors for all cases are listed in **Table 4-2** and **Table 4-3**.

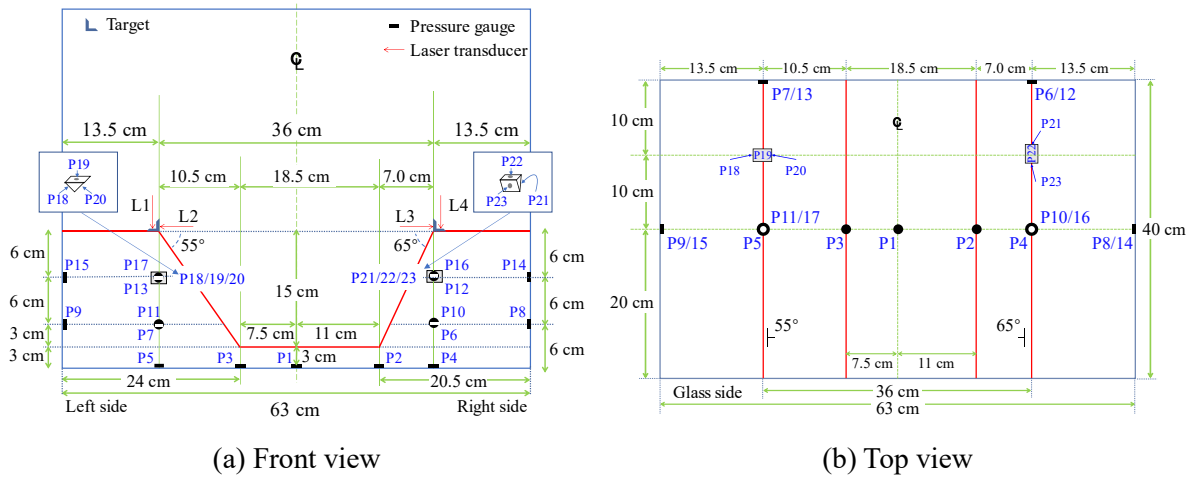


Fig. 4-14 Configuration of sensors in Case 1

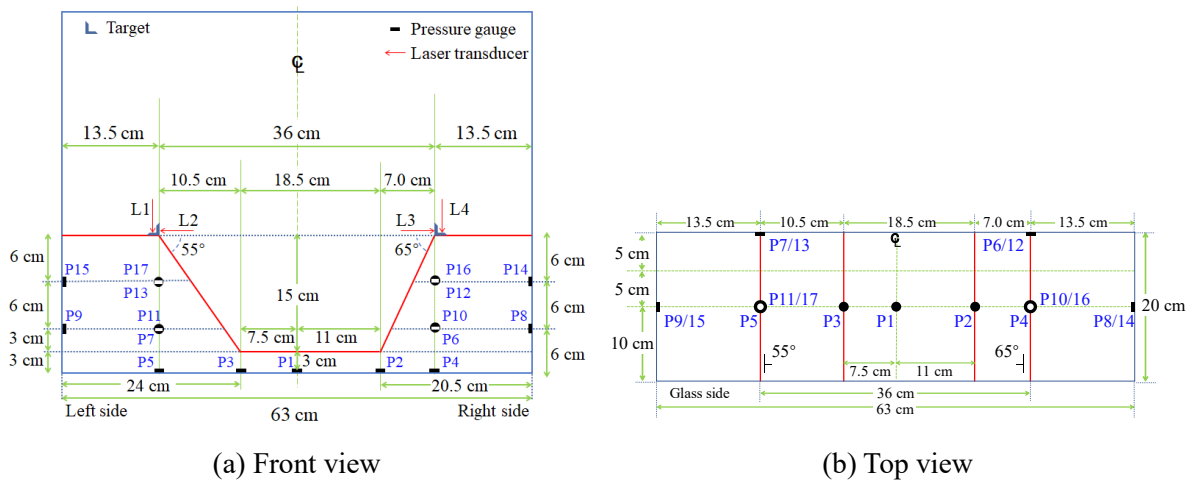


Fig. 4-15 Configuration of sensors in Case 2

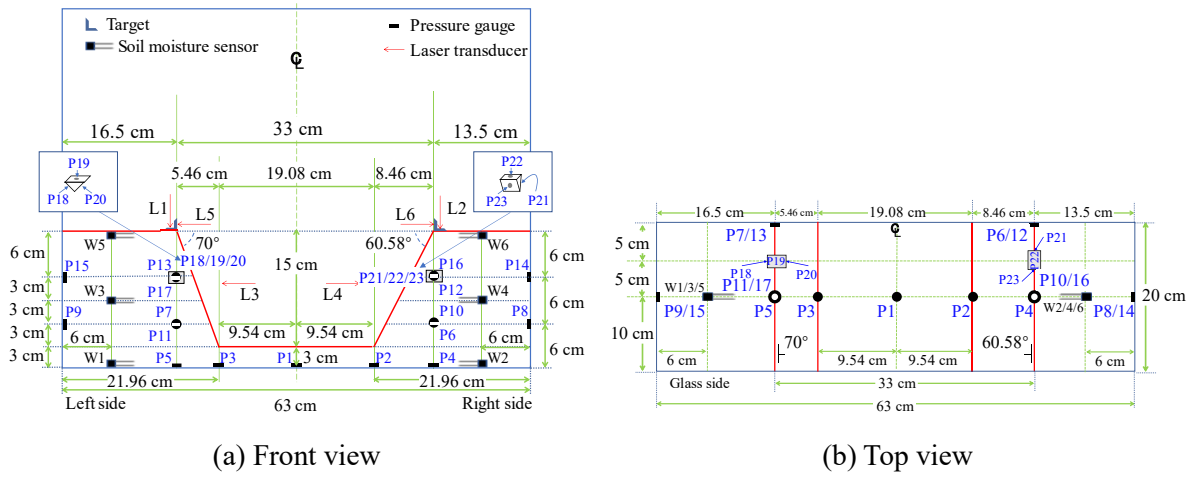


Fig. 4-16 Configuration of sensors in Case 3

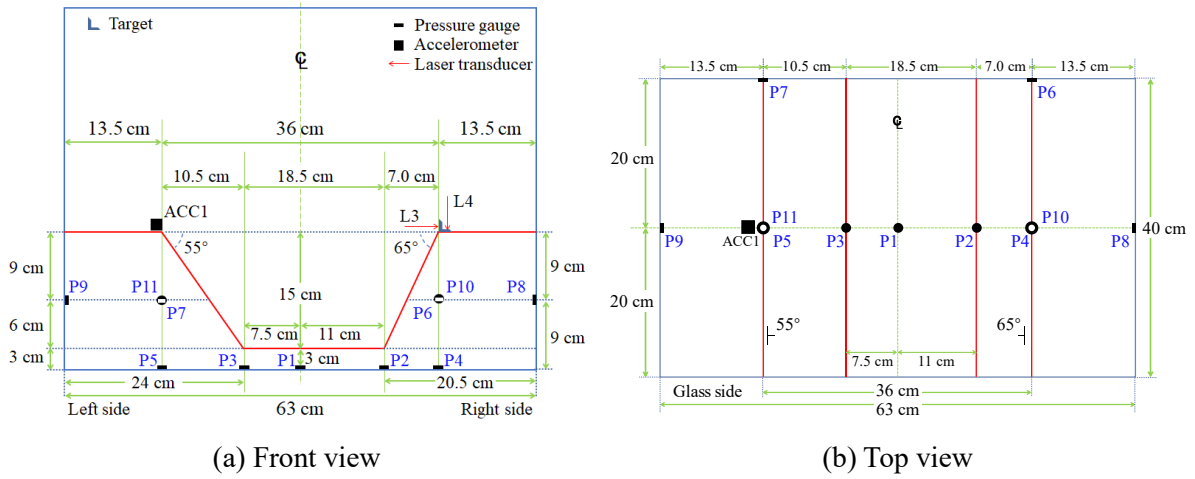


Fig. 4-17 Configuration of sensors in Case 4

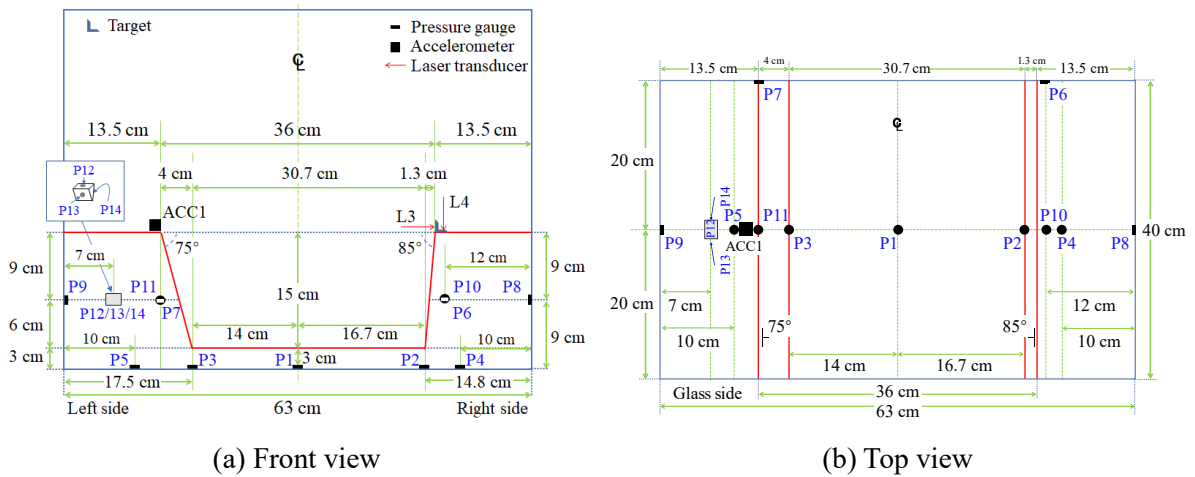


Fig. 4-18 Configuration of sensors in Case 5

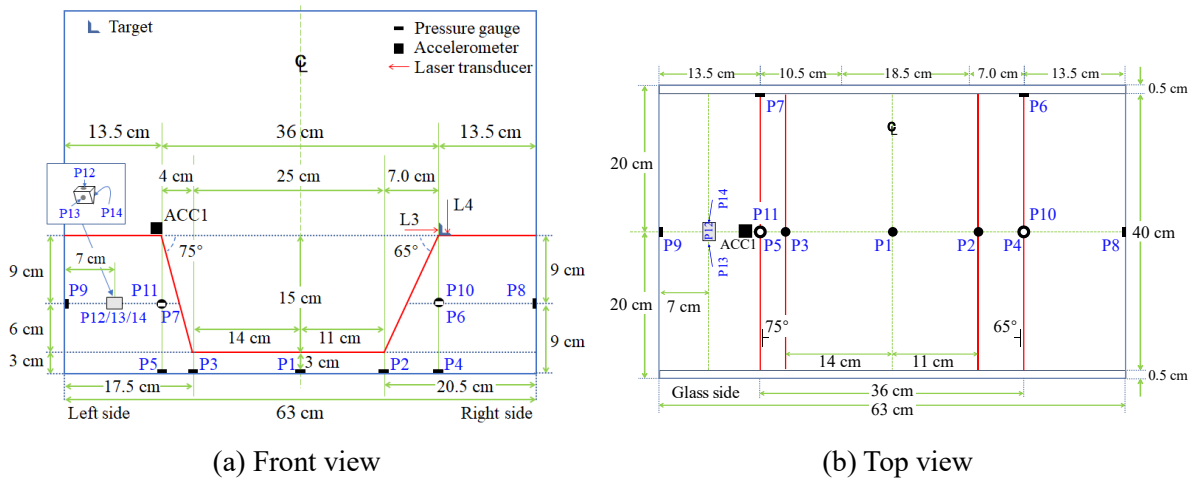


Fig. 4-19 Configuration of sensors in Case 6

Table 4-2 List of earth pressure sensors for all cases

No.	Date	Earth pressure sensor																						
		P 1	P 2	P 3	P 4	P 5	P 6	P 7	P 8	P 9	P 10	P 11	P 12	P 13	P 14	P 15	P 16	P 17	P 18	P 19	P 20	P 21	P 22	P 23
1	220704	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
2	220513	○	○	○	○	○	○	○	○	○	○	△	○	○	○	○	○	○	○	○	○	○	○	○
3	230220	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
4	210805	○	○	○	○	○	○	○	△	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
5	210921	○	○	○	○	○	○	○	△	○	○	△	○	○	○	○	○	○	○	○	○	○	○	○
6	211007	△	○	○	○	○	△	△	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned
 - The sensor was not placed

Table 4-3 List of other sensors

No.	Date	Laser sensor						Moisture sensor						Accelerometer
		LD 1	LD 2	LD 3	LD 4	LD 5	LD 6	W 1	W 2	W 3	W 4	W 5	W 6	
1	220704	○	○	○	○	○	○	○	○	○	○	○	○	○
2	220513	○	○	○	○	○	○	○	○	○	○	○	○	○
3	230220	○	○	○	○	○	△	○	○	○	○	○	○	○
4	210805	○	○	○	○	○	○	○	○	○	○	○	○	○
5	210921	○	○	○	○	○	○	○	○	○	○	○	○	○
6	211007	○	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned
 - The sensor was not placed

4.2.3 Test procedure

The experimental procedure of every case followed a similar approach. First, dry Hiroshima sand was blended with water to achieve a desired water content ($w=8\%$). Prepared soil was then divided into six equal portions and compacted into the soil chamber in six layers using a wooden hammer, with each layer being 3 cm thick. The weight required for each layer was obtained according to the compaction degree ($D_c=80\%$ or $D_c=90\%$) and the volume of each layer. Sensors were placed once the soil was compacted to the required heights. After forming a horizontal foundation with a height of 18 cm, excess soil was excavated from the center, and two slope surfaces were shaped using a spatula. The preparation process is shown in **Fig. 4-20**. For each case, the length of slope crests, indicated by black solid line, remained constant, while the slope angle was altered.

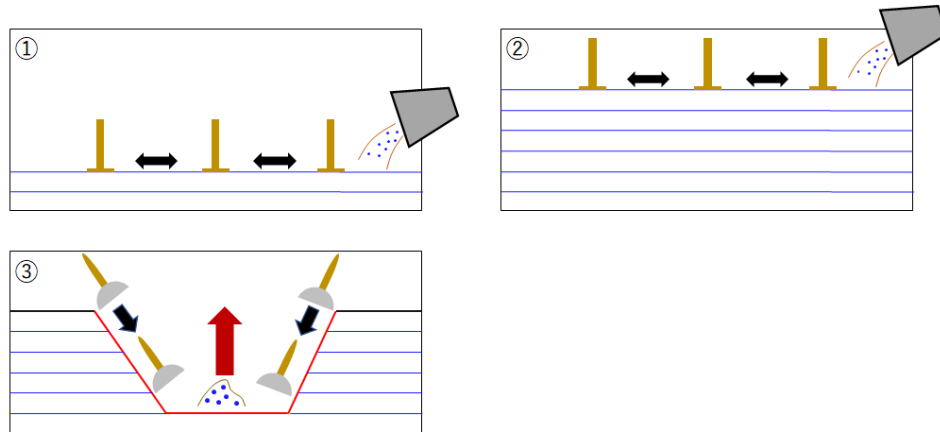


Fig. 4-20 Flow chart of the preparation for a slope model

Built model was then installed onto the centrifuge platform for testing. For $D_c=80\%$, the centrifugal loading was initially increased steadily from 1G to 50G. Then, the loading was increased in steps of 10G for safety and recording convenience. Once the centrifuge reached the maximum loading of 80G, it was stopped, and the centrifugal acceleration was gradually decreased back to 1G for unloading. In case 3, since failure occurred within 50G, the test was stopped at that point. For $D_c=90\%$, the centrifugal loading was also initially increased to a certain value. Then, the loading was increased in steps of 10G. In Case 4 and Case 5, the loading was applied twice for safety. In Case 6, the model was accelerated to 100G directly. The detailed loading process for each case is presented in **Table 4-1**.

4.3 Experimental Results

In all cases, the slope sliding process was captured by a high-speed camera. All sliding events occurred within one second, with time-ordered images provided in this section. Additionally, stress and displacement measurements from installed sensors are presented below.

4.3.1 Laterally confined steep uniform slope under static loading ($D_c=80\%$)

4.3.1.1 Case 1 ($\beta_L=55^\circ, \beta_R=65^\circ$, model breath =40 cm)

Sliding process

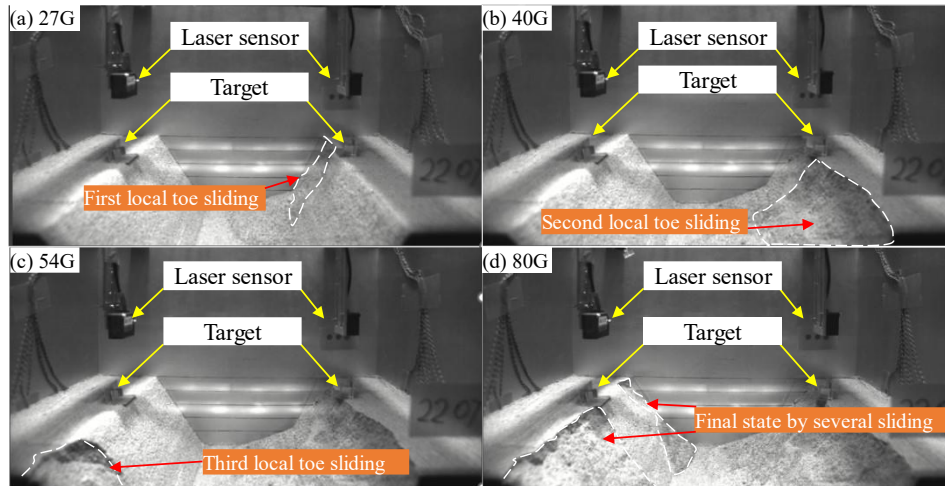


Fig. 4-21 Sliding process in Case 1: (a) first local toe sliding on the right slope; (b) second local toe sliding on the right slope; (c) third local toe sliding on the left slope; (d) final state of the left slope

The definitions of different types of sliding are clarified as follows:

Toe Sliding: The sliding surface connects the slope crest and the toe across the entire slope breadth.

Local Toe Sliding: The sliding surface connects the slope crest and the toe but only across partial breadth.

Local Sliding: The sliding surface does not connect the slope crest and the toe.

Corner sliding: Sliding occurs at the convex corner of the slope (slopes with boundary gaps).

In Case 1, the first local toe sliding happened on the right-hand side slope at 27G (see **Fig. 4-21 (a)**), near the back wall. At 40G, this sliding surface extended along the crest by another local failure (see **Fig. 4-21 (b)**). On the left slope, the first local sliding occurred at 54G (see **Fig. 4-21 (c)**), followed by several times of small-scale local sliding, but till the maximum centrifugal loading in this case (80G), the central part of the left slope remained stable (see **Fig. 4-21 (d)**).

Displacement

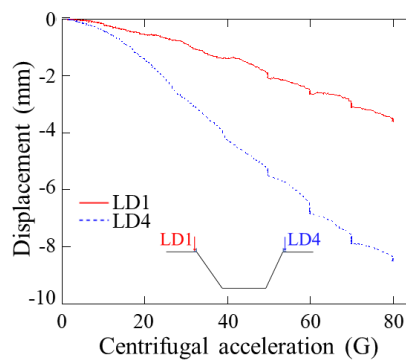


Fig. 4-22 Vertical displacement of slope crests in Case 1 with increasing centrifugal acceleration

The value of vertical displacement of both crests decreases linearly with centrifugal acceleration (see **Fig. 4-22**). Negative values indicate that targets move away from the laser transducer, which means settlement of the slope crest.

Earth pressure

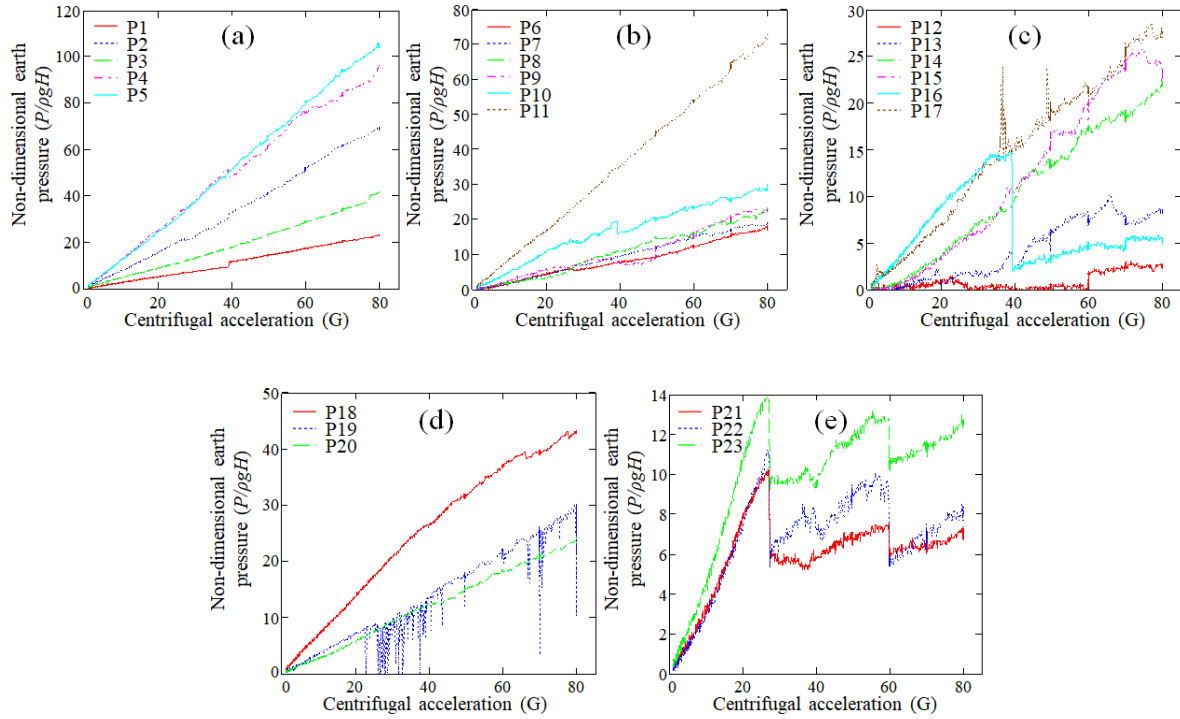


Fig. 4-23 Measured earth pressure with increasing centrifugal acceleration in Case 1: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

The earth pressure (P) is normalized using $\rho g H$ (ρ : total density of humid sand, g : gravitational acceleration constant; and H : model height). These symbols have the same meaning across the following cases. **Fig. 4-23(a)** to **Fig. 4-23(c)** show the non-dimensional earth pressures at three different heights of the model, except two three-directional earth pressure gauges, which are shown in **Fig. 4-23 (d)** and **Fig. 4-23 (e)**. They all exhibit an overall increasing trend with occasional drops, indicating stress release due to sliding.

4.3.1.2 Case 2 ($\beta_L=55^\circ$, $\beta_R=65^\circ$, model breath =20 cm)

Sliding process

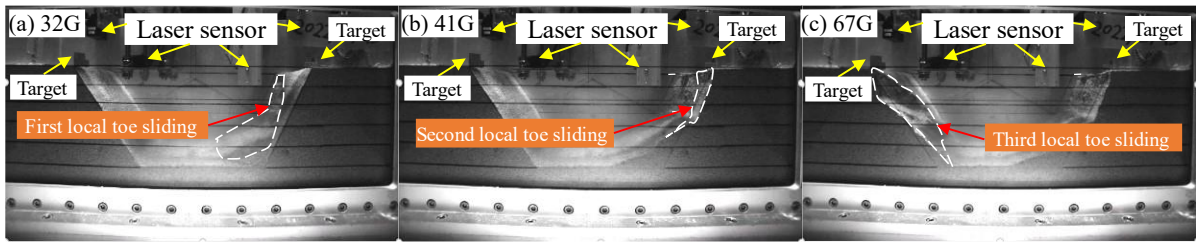


Fig. 4-24 Sliding process in Case 2: (a) first local toe sliding on the right slope; (b) second local toe sliding on the right slope; (c) third local toe sliding on the left slope

In Case 2, local toe sliding was observed twice on the right slope between 1G and 50G, at 32G and 41 G, respectively (see **Fig. 4-24 (a) and (b)**). On the left slope, local toe sliding occurred once at 67G (see **Fig. 4-24 (c)**). Compared to Case 1, this case, with the same slope cross-section but a different model breadth, demonstrated greater stability.

Displacement

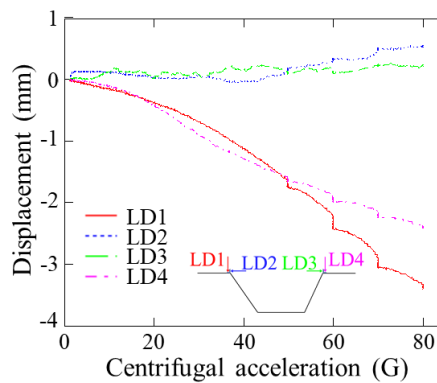


Fig. 4-25 Vertical and horizontal displacement of slope crests in Case 2 with increasing centrifugal acceleration

In this case, settlement magnitudes measured by LD1 and LD4 on both slopes are similar, developing steadily with increasing centrifugal acceleration (see **Fig. 4-25**). The values recorded by the other two laser sensors, LD2 and LD3, which monitor the slope crests' horizontal displacement, rise gradually with centrifugal loading, indicating that both crests are moving toward the center of the model.

Earth pressure

Similar to **Fig. 4-23**, **Fig. 4-26** shows the correlation between non-dimensional earth pressures and the centrifugal loading at three different heights of the slope model in Case 2. The pattern is similar to Case 1, with an increasing trend and some drops.

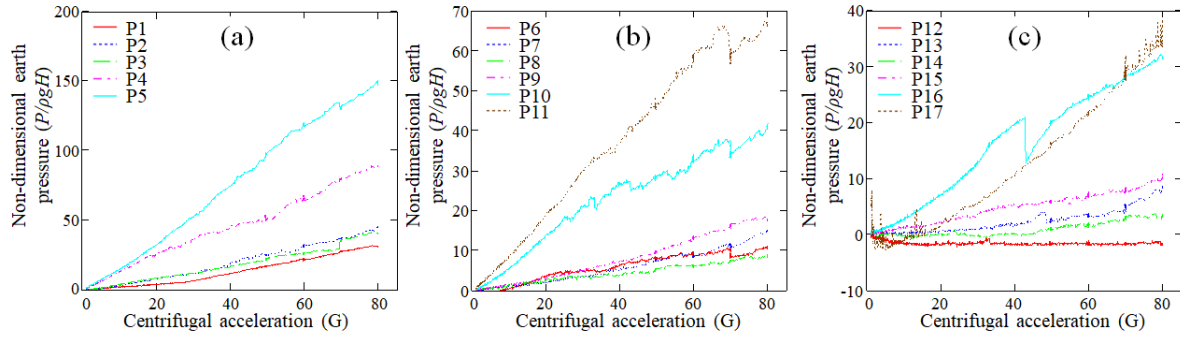


Fig. 4-26 Measured earth pressure with increasing centrifugal acceleration in Case 2: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height

4.3.1.3 Case 3 ($\beta_L=70^\circ$, $\beta_R=60.58^\circ$, model breath =20 cm)

Sliding process

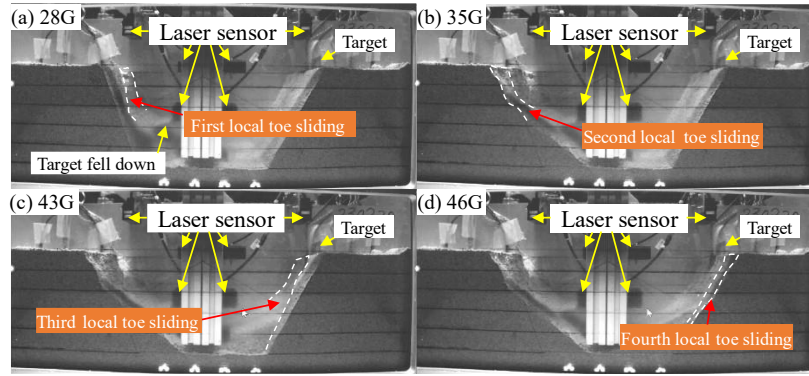


Fig. 4-27 Sliding process in Case 3(a) first local toe sliding on the left slope; (b) second local toe sliding on the left slope; (c) third local toe sliding on the right slope; (d) fourth local toe sliding on the right slope

In Case 3, first local toe sliding happened near the back wall on the left slope at 28G (see **Fig. 4-27 (a)**). A complete toe sliding surface was formed with another local toe sliding at 35G (see **Fig. 4-27 (b)**). The right slope remained stable until 43G, when a local toe sliding occurred near the back wall (see **Fig. 4-27 (c)**). Finally, a local toe sliding happened at 46G, leading to a complete siding surface on the right slope (see **Fig. 4-27 (d)**).

Displacement

In **Fig. 4-28**, abrupt value changes of LD1 and LD5 at 28G were caused by the falling of the target on the left slope during the first local toe sliding, while the change in LD3 at that moment, monitoring the left slope surface's horizontal displacement, was caused by sliding soil (see **Fig. 4-27 (a)**). Similarly, the increase in LD4 at 43G was induced by the sliding soil during the third local toe sliding (see **Fig.**

4-27 (c)).

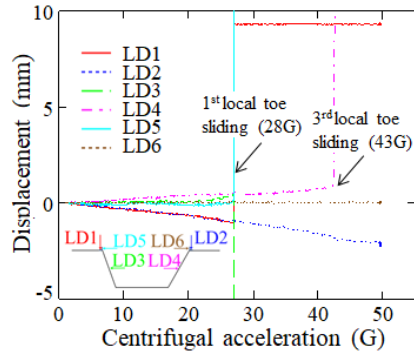


Fig. 4-28 Slope crests' vertical and horizontal displacement, and slope surfaces' horizontal displacement in Case 3 with increasing centrifugal acceleration

Earth pressure

Similar to Fig. 4-23, Fig. 4-29 presents the correlation between non-dimensional earth pressures and the centrifugal loading at three different heights of the slope model in Case 3.

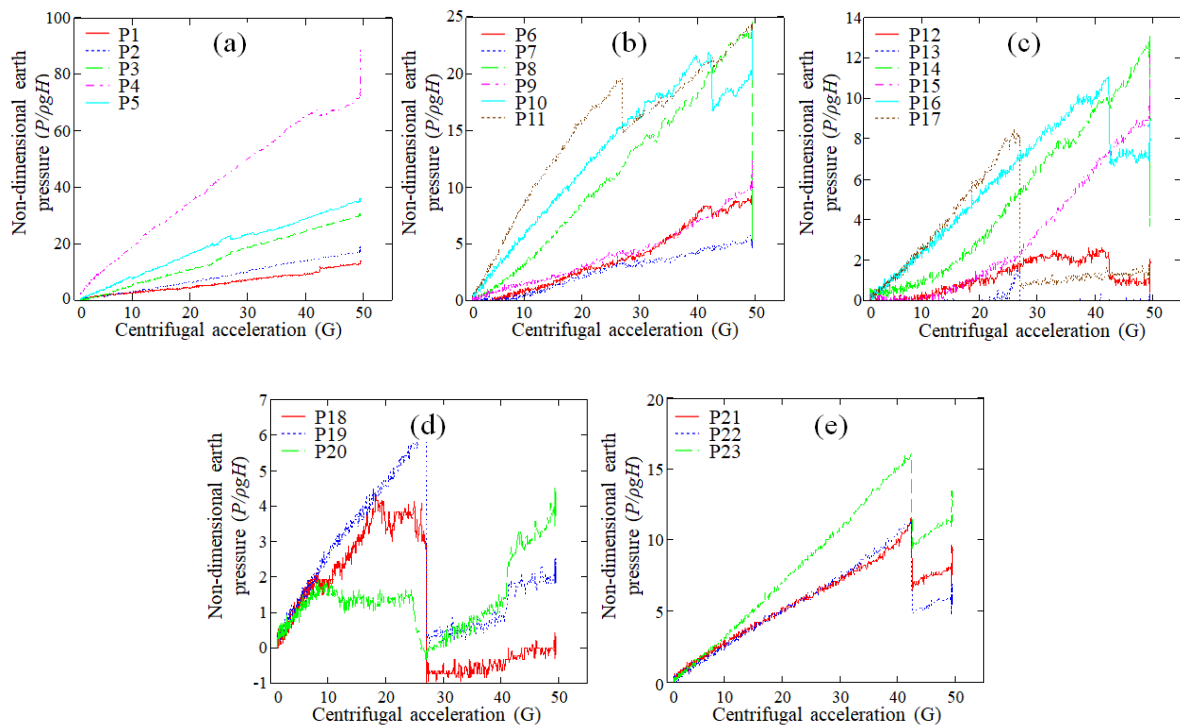


Fig. 4-29 Measured earth pressure with increasing centrifugal acceleration in Case 3: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

4.3.2 Laterally confined steep uniform slope under static loading ($D_c=90\%$)

4.3.2.1 Case 4 ($\beta_L=55^\circ, \beta_R=65^\circ$, model breath =40 cm)

Sliding process



Fig. 4-30 Model state after centrifugal loading in Case 4

In Case 4, both model slopes remained stable until the end of the experiment after two rounds of loading (see **Fig. 4-30**), due to high strength of damping material and the insufficient inclination of both sides.

Displacement

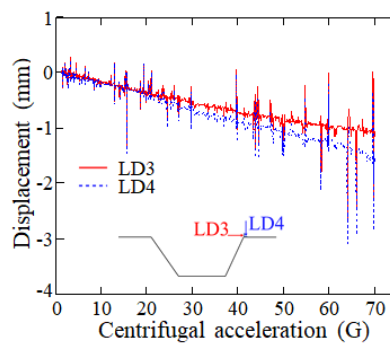


Fig. 4-31 The right slope crest' vertical and horizontal displacement during the first loading (70G) in Case 4

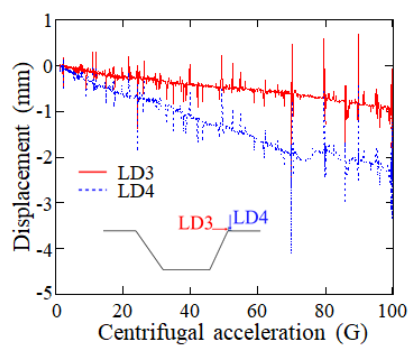


Fig. 4-32 The right slope crest' vertical and horizontal displacement during the second loading (100G) in Case 4

In this case, during both loading phases, the vertical displacement (LD4) of right slope crest reduces with centrifugal acceleration, indicating settlement. The negative value of the displacement in the horizontal direction (LD3) suggests that right slope crest moves away from the center of the slope. All displacements increase linearly with centrifugal acceleration (see Fig. 4-31 and Fig. 4-32).

Earth pressure

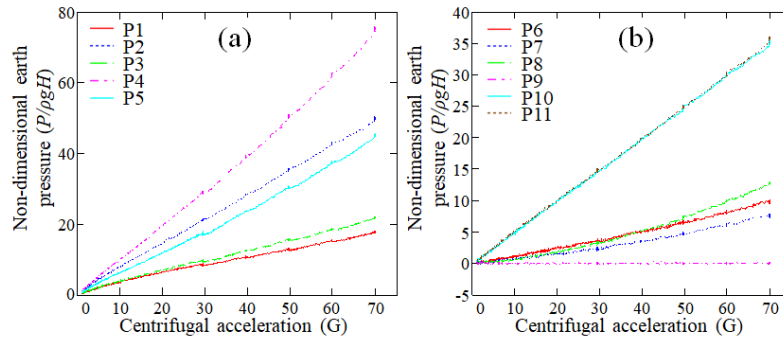


Fig. 4-33 Measured earth pressure with centrifugal loading during first loading (70G) of Case 4: (a) bottom earth pressure; (b) earth pressure at half height

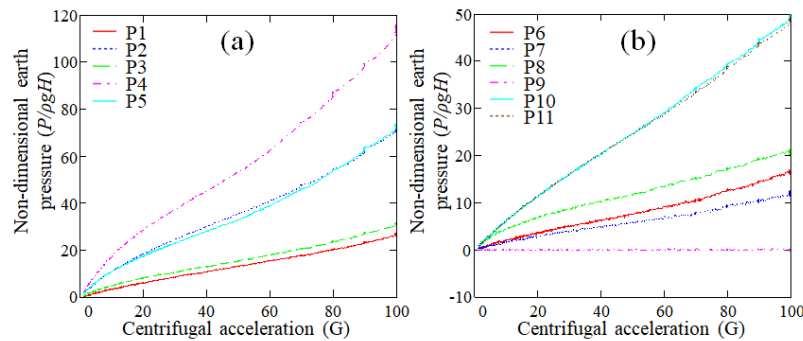


Fig. 4-34 Measured earth pressure with centrifugal loading during the second loading (100G) of Case 4: (a) bottom earth pressure; (b) earth pressure at half height

Fig. 4-33 and **Fig. 4-34** show the stress distribution in the model as centrifugal acceleration increases. All values rise steadily without abrupt changes, corresponding to the observed stable state of the model during the test. Additionally, sensor P9 malfunctioned in this case.

4.3.2.2 Case 5 ($\beta_L=75^\circ$, $\beta_R=85^\circ$, model breath =40 cm)

Sliding process

In Case 5 (see **Fig. 4-35**), three instances of local toe sliding occurred on the right slope at below 30G, 41G, and 69G during the first loading phase, which reached up to 70G. The sliding area gradually

extended from the corner near the back wall toward the glass side. The second loading phase, which reached a higher centrifugal acceleration, did not trigger any new sliding events.

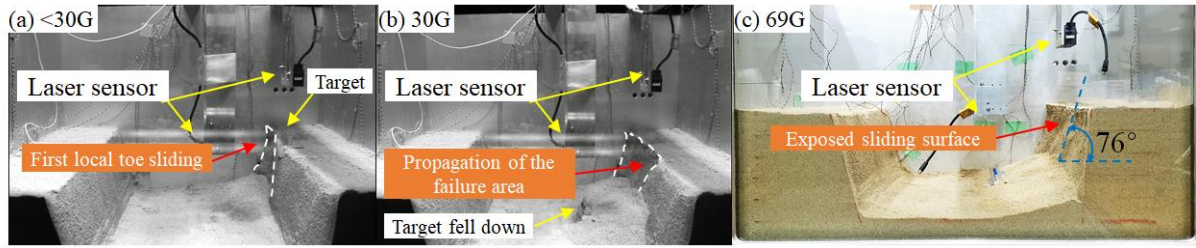


Fig. 4-35 Sliding process in Case 5

Displacement

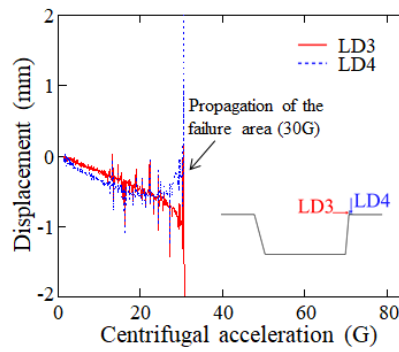


Fig. 4-36 The right slope crest's Vertical and horizontal displacement during the first loading (80G) in Case 5

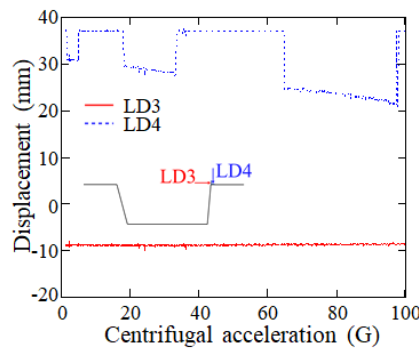


Fig. 4-37 The right slope crest's Vertical and horizontal displacement during the second loading (100G) in Case 5

In this case, all sliding of the right slope occurred during the first loading phase, which reached 70G. Since 30G during the first loading, the readings from two laser sensors (LD3 and LD4) have been out of range due to the target falling at that point (see **Fig. 4-36** and **Fig. 4-37**).

Earth pressure

Fig. 4-38 and **Fig. 4-39** show the variation of non-dimensional earth pressure with centrifugal acceleration during two loading phases in Case 5. This data was collected using two layers of earth pressure sensors at different heights in the model and a three-dimensional gauge positioned at half the height of the left slope. During the first loading phase, stress release due to sliding was recorded by sensors P6 and P10 at 30G. In this case, sensors P8 and P12 malfunctioned.

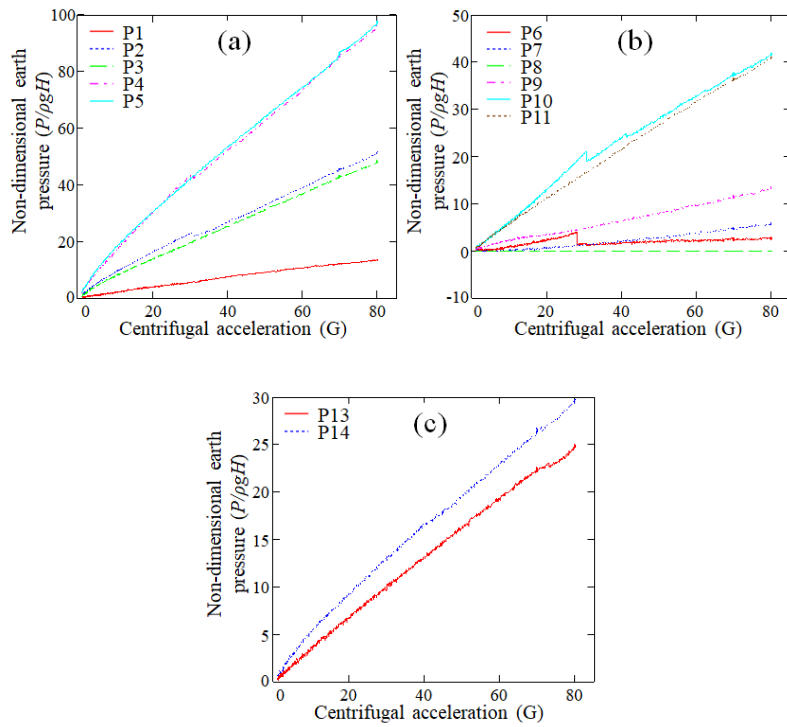
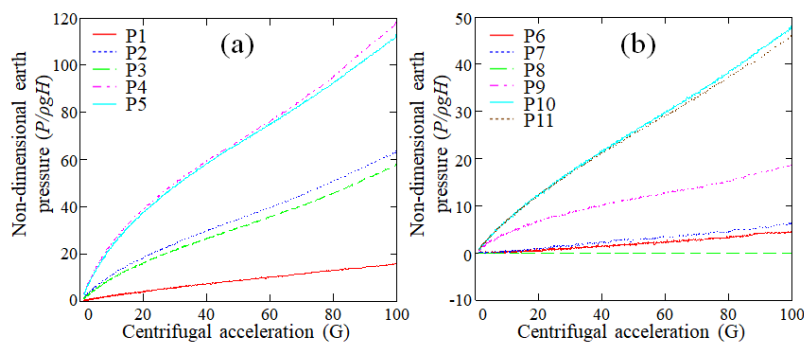


Fig. 4-38 Measured earth pressure with centrifugal loading during first loading (80G) of Case 5: (a) bottom earth pressure; (b) earth pressure at half height; (c) earth pressure at the three-directional gauge in the left slope



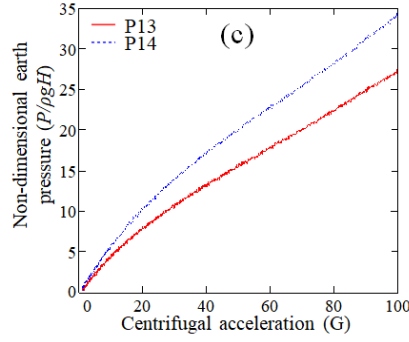


Fig. 4-39 Measured earth pressure with centrifugal loading during second loading (100G) in Case 5: (a) bottom earth pressure; (b) earth pressure at half height; (c) earth pressure at the three-directional gauge in the left slope

4.3.2.3 Case 6 ($\beta_L=75^\circ$, $\beta_R=65^\circ$, model breath =39 cm)

Sliding process

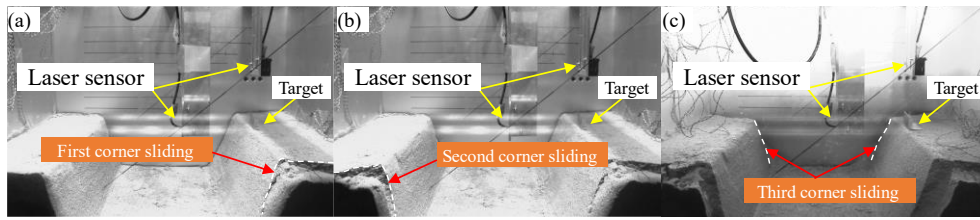


Fig. 4-40 Sliding process in Case 6

In Case 6 without lateral support (see **Fig. 4-7 (f)**), the sliding pattern was different from other cases where the corner sliding was dominant. The first corner sliding occurred at the glass side on the right slope (**Fig. 4-40 (a)**) followed by one happened on the left slope (see **Fig. 4-40 (b)**). Then two corners near the back wall collapsed at last (see **Fig. 4-40 (c)**). The central part of two slopes did not fail during the test. In this case, the corner sliding of slope boundaries without lateral support can be seen as failures of vertical slopes.

Displacement

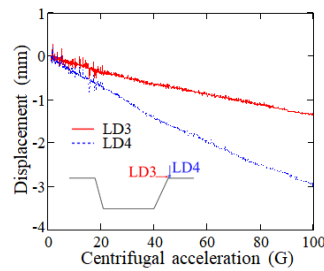


Fig. 4-41 The right slope crest's vertical and horizontal displacement in Case 6

Although corner sliding occurred in Case 6, the center of the slope crest remained stable throughout the test. The displacement measured by the laser sensors increased steadily with centrifugal acceleration, without any abrupt changes.

Earth pressure

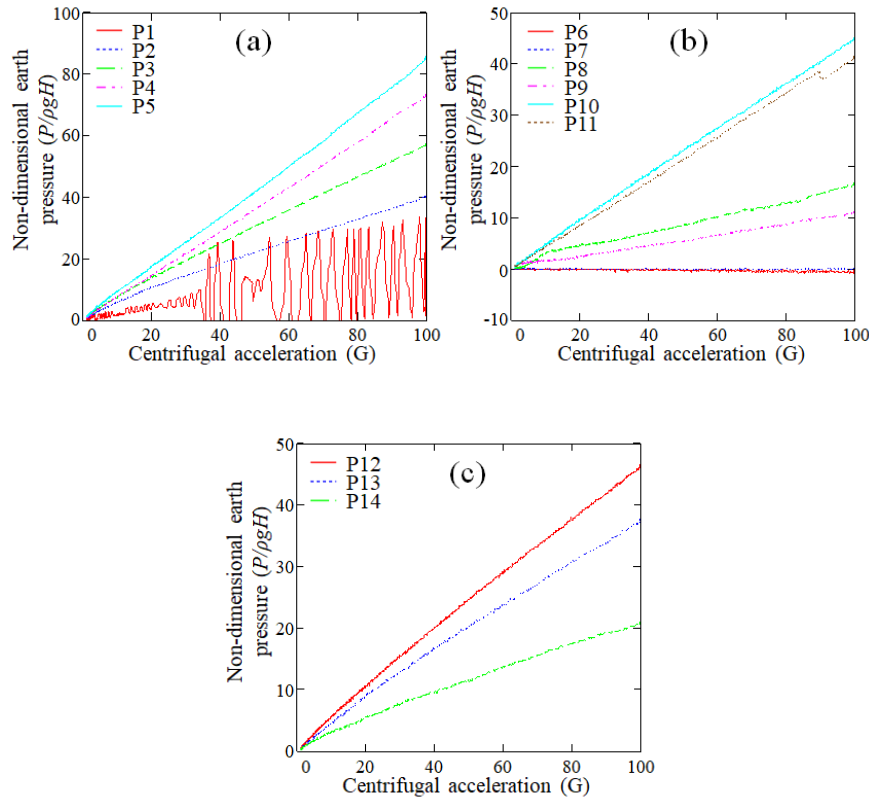


Fig. 4-42 Measure earth pressure with increasing centrifugal acceleration in Case 6:
(a) bottom earth pressure; (b) earth pressure at half height; (c) earth pressure at the three-directional gauge in the left slope

Fig. 4-42 shows the variation in non-dimensional earth pressures with centrifugal acceleration in the model for Case 6. Few drop points are observed, indicating that the stress release from corner sliding was not captured by the pressure sensors along the centerline. The values of P6 and P7 remained at zero, as they were positioned on the boundary to record horizontal earth pressure between the model and the back wall of the soil chamber. This lack of lateral support resulted from a gap near the wall (see **Fig. 4-19**).

The experimental outcomes from all cases are summarized in **Table 4-4**. Following the experiment, water content in model slopes was measured through collecting samples from three distinct heights of the model.

Table 4-4 Experimental outcomes of uniform slopes

No.	Date	Final water content (%)			Sliding (G)			
		Bottom	Middle	Top	Left		Right	
		(0-2 cm)	(8-10 cm)	(16-18 cm)	First	Last	First	Last
1	220704	8.7	7.5	5.8	54	80	27	40
2	220513	9.3	7.9	6.2	67		32	41
3	230220	Δ	7.3	6.2	28	35	43	46
4	210805	12.1	6.9	4.2	—	—	—	—
5	210921	Δ	Δ	Δ	—	—	<30	69
6	211007	Δ	Δ	Δ	Δ	Δ	Δ	Δ

Note: Δ The data was not measured – The data did not exist

4.4 Discussion

4.4.1 Stability and failure patterns of uniform slopes

Experimental outcomes indicate that slope angle and degree of compaction significantly influence slope stability and failure mode. Slopes with smaller angles were notably more stable than steeper slopes, with the first sliding event occurring at a higher centrifugal acceleration.

In the centrifugal model, only a portion of the slope failed during the first sliding event, differing from the plane strain assumption typically adopted in 2D slope instability analysis. The discrepancy is attributed to manual compaction of the centrifugal model, which led to inevitable inhomogeneity. In the model, the boundary part was less compacted than the central part. However, multiple sliding events at different levels of centrifugal loading often followed the same sliding surface.

Increasing the degree of compaction significantly enhances slope stability. When the compaction degree reached 90%, the slope model maintained a nearly vertical inclination even at high centrifugal acceleration.

4.4.2 Overburden pressure at the bottom

Five earth pressure sensors (P1 to P5) were positioned in each case, along the longitudinal centerline at the base of the model to measure variations in overburden pressure with centrifugal loading. Sensor P1 was placed under the center of the model, while P2 and P3 were located vertically below the slope toes. Sensors P4 and P5 were positioned vertically beneath the slope crests.

Fig. 4-43 compares the normalized overburden height through, calculated through back-calculation before (5G prior to the first sliding) and after sliding (at the peak loading), with the normalized actual overburden height (the slope outline). The overburden height is back analyzed through dividing the measured earth pressure, P , by $n\rho_t g$, where ρ_t denotes the total unit weight, n is the scaling factor in centrifuge modeling to account for gravity, and g represents gravitation acceleration. The overburden height is then normalized by the soil chamber's height, H , which is 47 cm for the wide soil chamber

and 30 cm for the narrow soil chamber. The distance from the center, d , along the x -axis is normalized by the soil chamber's width, L , which is 63 cm.

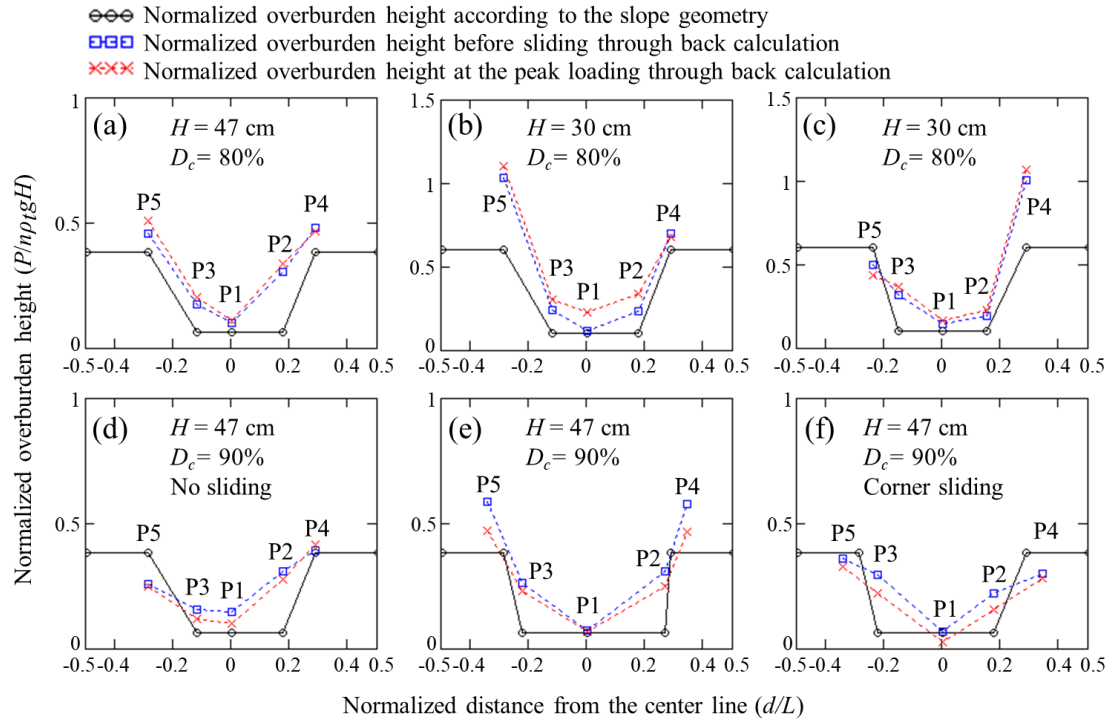


Fig. 4-43 Normalized overburden height at bottom of the model before sliding and at the peak loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6

Results show that back calculated overburden height under the center of the model (P1) before sliding closely matched the actual overburden height. However, at other positions, back calculated values during stable slope conditions deviated from the actual heights, with back-calculated values generally being higher. This difference was particularly noticeable under the slope toes (P2 and P3). Although P1, P2 and P3 had the same overburden height in geometry, the pressure under the steeper slope's toe was larger. It indicates that steep slopes induce higher stress concentration near the slope toe.

Additionally, for the first three cases with a degree of compaction (D_c) of 80%, the normalized overburden pressures before sliding were lower than those after sliding. In contrast, for the last three cases with a compaction degree of 90%, the normalized overburden pressures before sliding were greater than those observed at peaking loading.

4.4.3 Comparison between experiments and the 2D theory

Experimental outcomes are compared with the 2D theory as follows. The slope models' conditions in the centrifugal experiments are similar to those of most open-pit mine slopes, which are newly constructed and temporary. Therefore, peak strength, instead of residual strength, of the material is used for prediction.

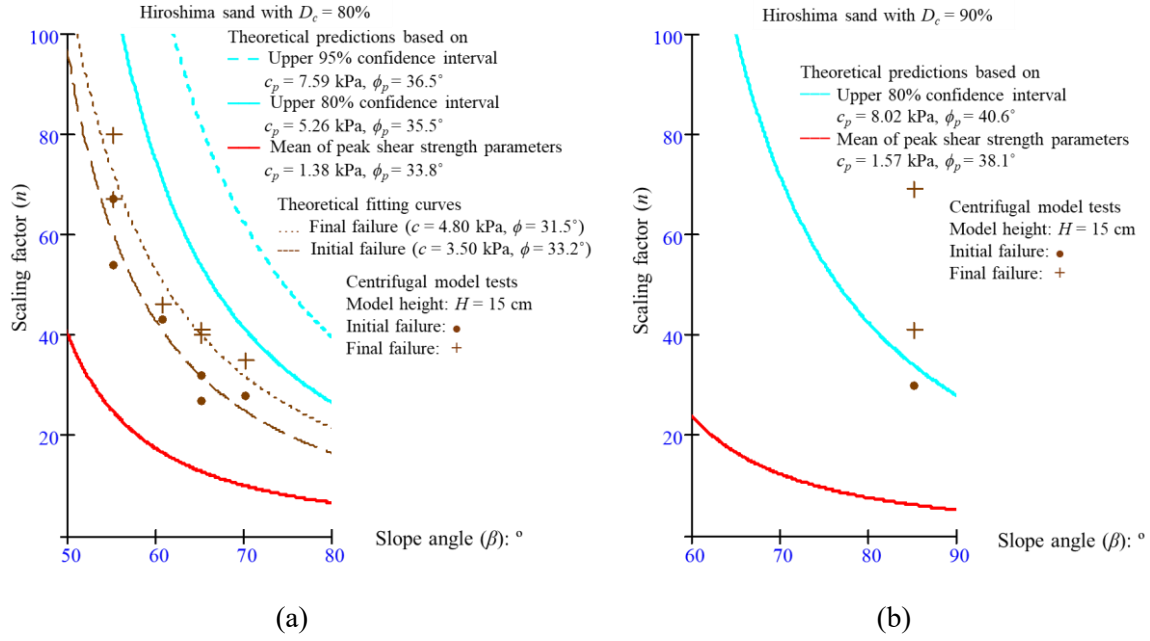


Fig. 4-44 Centrifugal model tests and theoretical predictions from the mean and the upper range of parameters for Hiroshima sand with water content of 6%-10%: (a) 80% compaction degree; (b) 90% compaction degree

The concept of confidence interval (CI) is employed to explain uncertainties in cohesion caused by variations in water content distribution and limitations in DST measurement. First, the water content in the model slopes ranged from 6% to 10%, varying by locations (see **Table 4-4**). Different water contents result in varying soil shear strengths, particularly in cohesion. Second, in centrifugal modeling, the normal stress on the failure plane is lower than 20 kPa, while as mentioned in **Chapter 3**, the direct shear apparatus used has a limitation in that it cannot apply normal pressures below 40 kPa. As a result, the absence of data introduces uncertainty in the slope (friction angle) of the strength envelope for soils with different water contents. Since the actual cohesion is low, ranging from 0 to 10 kPa, even minor changes in the slope can lead to significant variability in the intercept (cohesion). Furthermore, the proposed theory is highly sensitive to cohesion, making its predictions particularly vulnerable to these uncertainties. To address this, it is essential to introduce confidence levels rather than just using the mean value. An 80% CI based on the entire DST dataset with varying water contents was selected to capture this uncertainty while avoiding an excessively wide prediction interval (see **Fig. 4-46**). The initial and final failures in centrifugal model tests are marked with dots and plus signs in **Fig. 4-44**. According to **Eq. (4-24)**, the curve on the lower boundary represents the predicted failure scaling factor relative to the slope angle, using the mean of peak shear properties. The upper curve employs strength parameters corresponding to the confidence interval, calculated by **Eq. (4-68)**, where z represents the confidence level value and SE is the standard error, determined from **Eq. (3-4)** and (3-5). **Fig. 4-44 (a)** presents upper bounds based on CI of 80% and 95%. The prediction becomes more conservative with a larger CI. While the choice of CI is somewhat arbitrary, its value takes 80% in this study.

$$CI = \text{mean} + z \cdot SE$$

(4-68)

All test results for the initial and final failures with D_c of 80%, and the initial failure with D_c of 90% fall within the range defined by boundary curves, supporting the accuracy of the theoretical prediction.

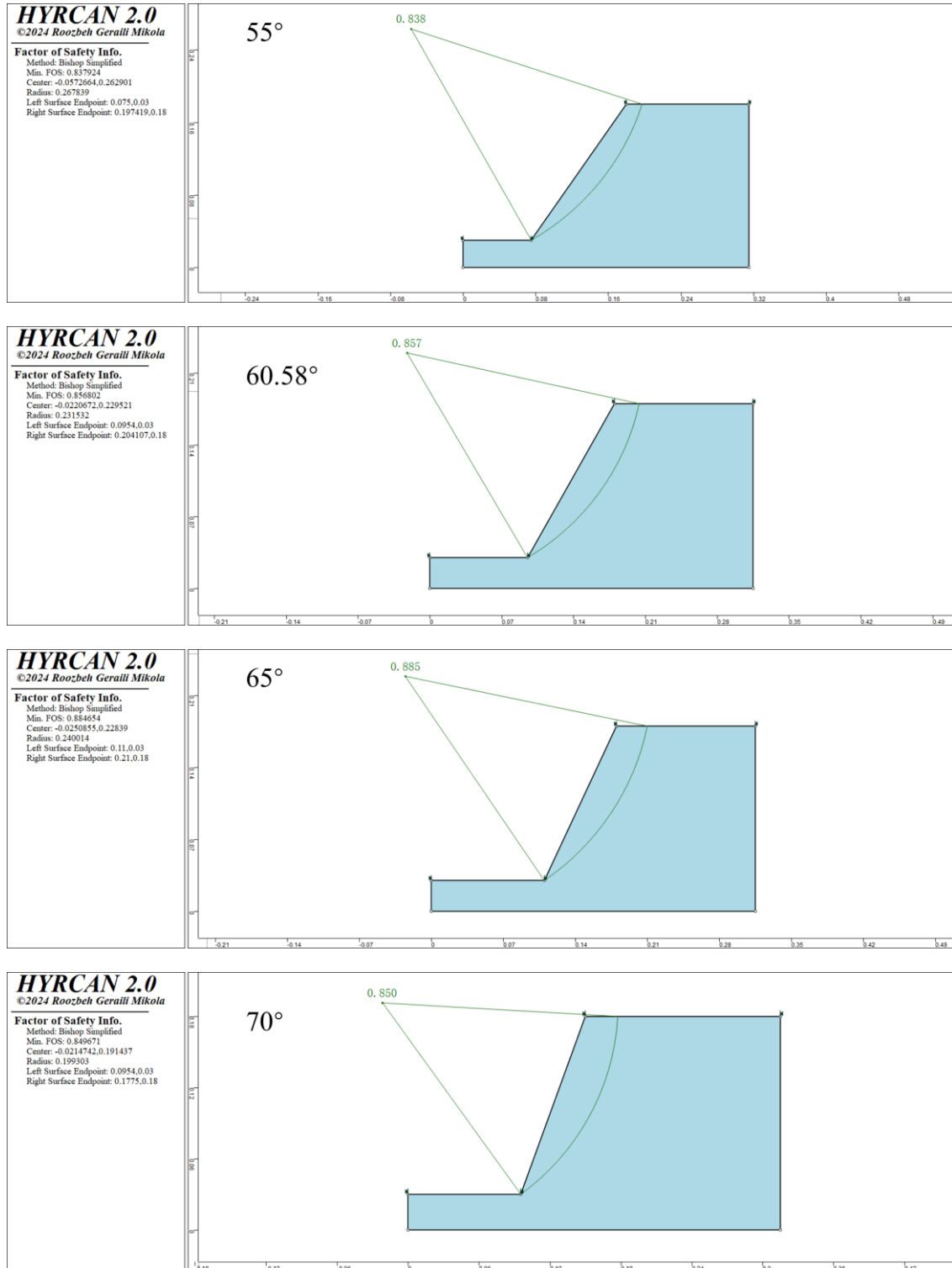


Fig. 4-45 FS calculated by Bishop Simplified method based on regression of the initial failures

For the group with $D_c = 80\%$, taking the strength parameters by regression on the initial failure ($c = 3.5$ kPa and $\phi = 33.2^\circ$), the results can be compared with Bishop Simplified method in HYRCAN, a free software in slope stability. As shown in **Fig. 4-45**, the obtained FS is below 1, indicating that the slice method is more conservative than the proposed theory, being consistent with **Fig. 2-1**.

The figures also present the back-calculated strength parameters obtained by regression for the initial and final failures with D_c of 80%. The back-calculated friction angle for initial failures is 0.98 times the mean peak value, while the cohesion is 2.5 times the mean peak value. It is noted that the curve gradually shifts from the lower to the upper boundary as cohesion increases, a trend not observed when adjusting the internal friction angle.

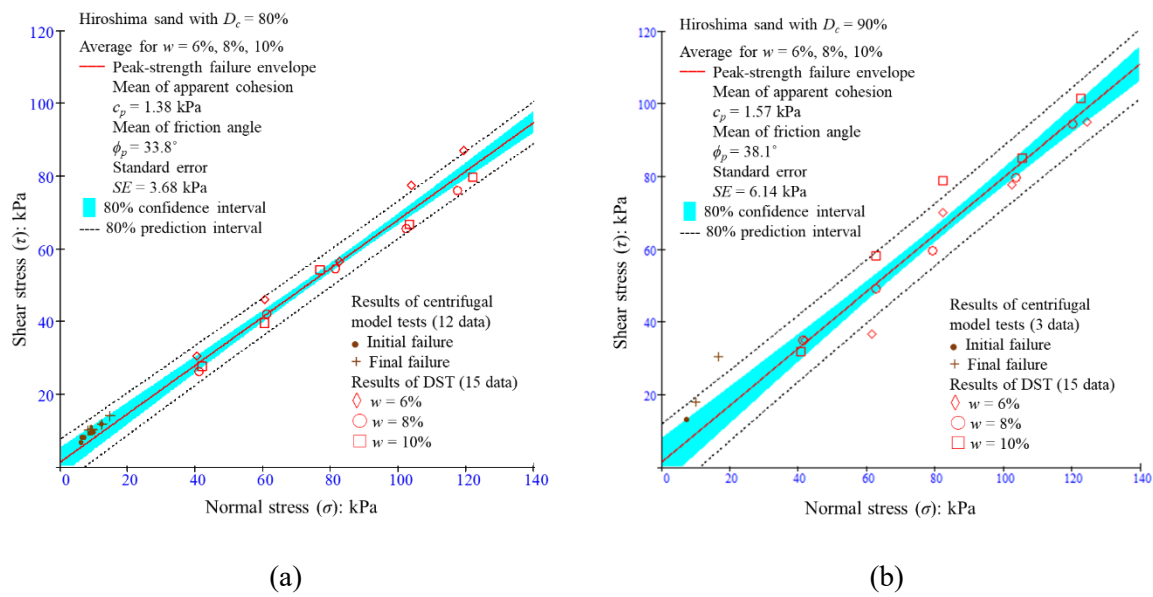


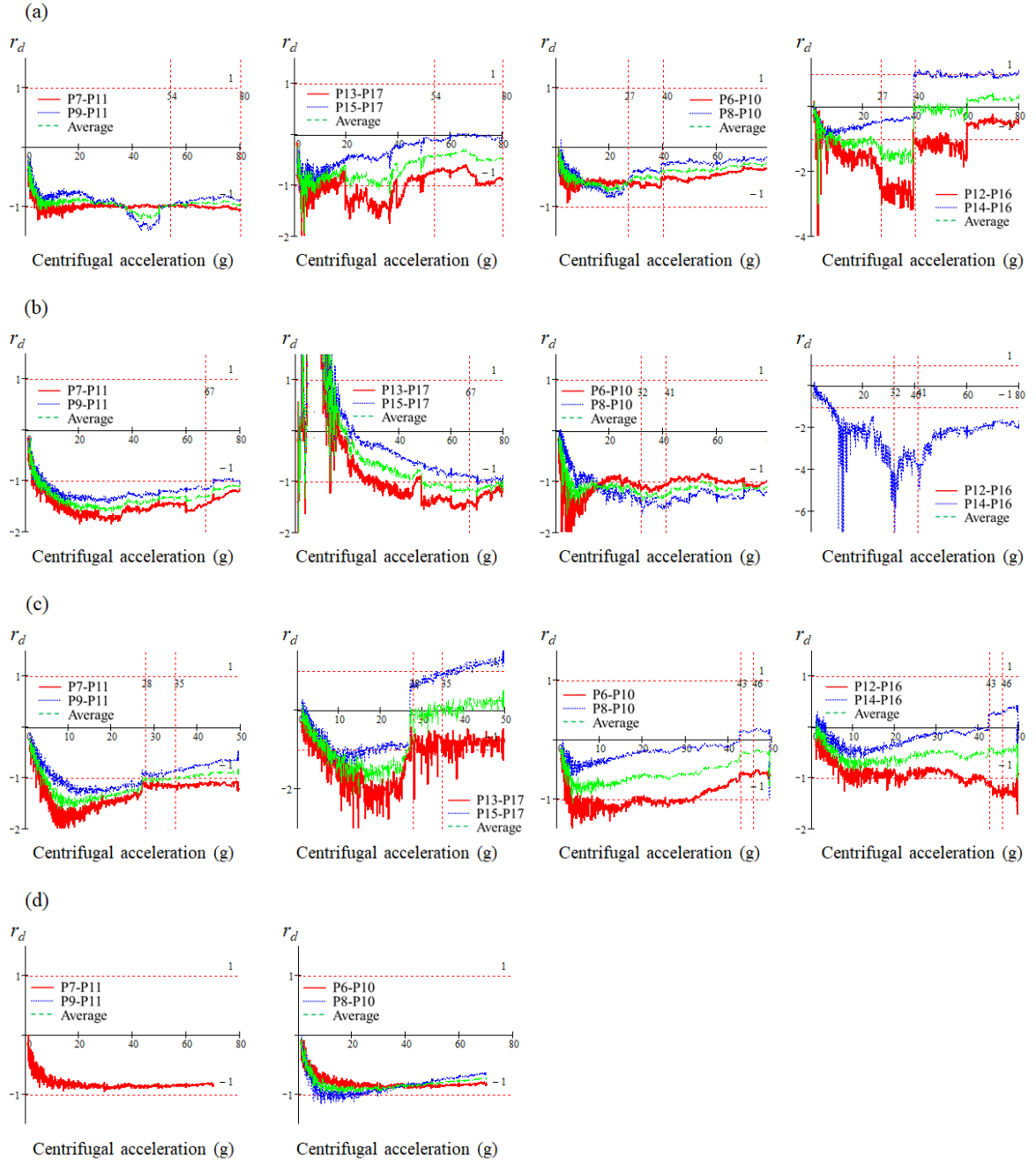
Fig. 4-46 Comparisons between centrifugal model tests, direct shear tests and the peak-strength failure envelope average for Hiroshima sand with water content of 6%-10%, plotted in the normal and shear stress space: (a) 80% compaction degree; (b) 90% compaction degree

The test results for the initial and final failure are plotted alongside the DST results in the normal and shear stress space (see **Fig. 4-46**) based on **Eq. (4-35)** and **(4-36)**. The 80% confidence interval for the failure envelope is represented by two dashed lines. The stress level in the experiment, which is below 20 kPa, is relatively lower than that in the DST. For $D_c = 80\%$, all test results fall within the prediction interval. For $D_c = 90\%$, the initial failure is within the interval with the exception of subsequent failures.

4.4.4 Variation of the slope state with increasing centrifugal loading

As mentioned in **section 4.1.3**, the reduction ratio, r_d , defined in **Eq. (4-45)** represents the slope state, ranging from an active state to a passive state. The model slopes deformed under the centrifugal loading during test, resulting in varied states. For every slope, two r_d values can be obtained for a certain height, yielding eight r_d values for models with $D_c = 80\%$ and four r_d for models with $D_c = 90\%$. For example,

in the right slope of Case 1, a vertical earth pressure sensor (P10 on the center line of the model) and two horizontal earth pressure sensors (P6 on the back wall, P8 on the right wall) are located 6 cm above the base of the soil chamber, hence, by combining P10 with P6 and P8, two r_d values are obtained (Tao & Pipatpongsa, 2023).



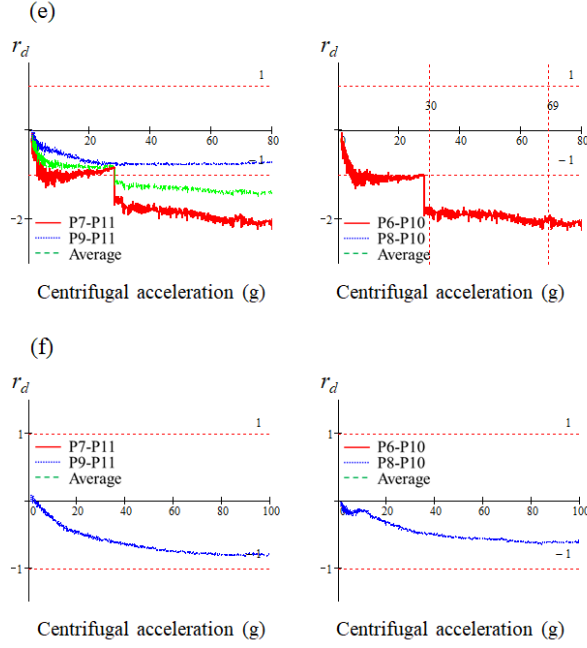


Fig. 4-47 Variation of reduction ratio with centrifugal loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6

Fig. 4-47 presents the variation of the reduction ratio, r_d , with increasing centrifugal acceleration. Due to the malfunction of sensors P12 in Case 2, P9 in Case 4, and P8 in Case 5, some curves could not be generated. Similarly, because of a clearance between the model and the boundary wall, sensors P6 and P7 in Case 6 recorded zero values, so the r_d values derived from these sensors were excluded.

Initially, each curve decreases from a positive initial value as centrifugal acceleration increases, indicating that the slope transitions from an isotropic or slightly passive state to an active state, thereby becoming increasingly unstable. In all plots except **Fig. 4-47 (d)**, the solid curve (P7-P11) is lower than the dotted curve (P9-P11), although both follow the same trend, suggesting that the slope deforms more near the back wall than near the left or right wall due to slope movement (Tao & Pipatpongsa, 2023). An average curve is created to account for this difference, providing a depiction of the slope state.

The vertical dashed lines in each figure indicate the centrifugal acceleration at which the initial and final sliding happened. The curves tend to show an abrupt jump around the sliding moment, suggesting that the slope attempts to return to an isotropic state after sliding. Comparing the curves of slopes with $D_c = 80\%$ (Case 1, 2 and 3) with that of slopes with $D_c = 90\%$ (Case 4, 5 and 6), the latter exhibit smoother curves. This indicates that increased compaction results in smoother curves. Most cases approach the limit active state ($r_d = -1$) by the final stage of testing.

4.4.5 Impact of lateral resistance considering 3D conditions

Considering the effect of lateral resistance, the FS of a slope under 3D conditions can be calculated by **Eq. (4-44)** and **Eq. (4-56)** once knowing the reduction factor (r_d). The stability of the slope increases as

r_d rises. The interface friction angle (ϕ_i) and adhesion (c_i) between the Hiroshima sand and the Aluminum wall of the soil chamber are 25.4° and 0.22 kPa, respectively (Wongchana, 2024).

The failure angle under 3D conditions is approximated using Eq. (4-23) for simplification. By setting the FS equal to 1, the correlation between the slope inclination and scaling factor is plotted, as shown in Fig. 4-48. As indicated by Eq. (4-57), the lateral resistance has an effect equivalent to increased internal cohesion. In the previous section, r_d decreases to -1 with increasing centrifugal loading, indicating that the model shows a limit active state where the lateral resistance is negligible. This condition nearly aligns with the curve based on 2D theory, as shown by the dashed line. Thus, for a uniform slope under static conditions, 2D theory is sufficient to estimate stability.

As r_d increases, the curve shifts upward. When r_d reaches 0.5, the curve aligns with the experiments, indicating that the slope state, especially in a passive state, can significantly influence its stability if the slope is laterally confined.

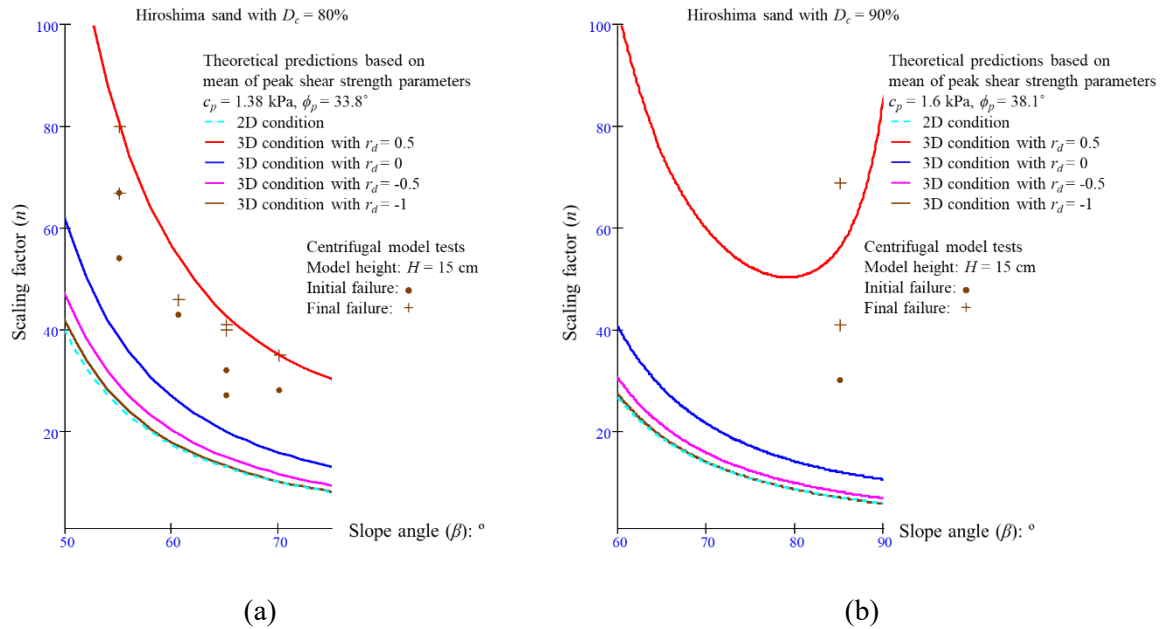


Fig. 4-48 Centrifugal model tests and theoretical predictions under 3D conditions considering the lateral resistance: (a) 80% compaction degree; (b) 90% compaction degree

4.4.6 Principal stress rotation and stress path measured by the three-directional sensor

As introduced in Section 4.2.2, the three-directional sensor enables direct measurement of stress components, hence the direction and magnitude of principal stresses, from earth pressures recorded in three distinct directions. The orientation provides critical insights into slope stability by revealing the stress distribution and potential failure mechanisms. If the direction is along the slope surface, it may indicate that the slope is experiencing movement or deformation, suggesting a destabilized or active state. If the stress direction is away from the slope face, it may suggest a resisting or passive state, indicating stability in that region. Areas with significant variations in the orientation are likely stress

concentrated zones, which can precede slope failure. Tracking the rotation and magnitude of the principal stress can reveal progressive failure mechanisms. Additionally, the force chain network represents the load transfer path and the structural framework that resists external loads (Yu et al., 2023). Understanding its configuration can inspire effective strengthening methods. Moreover, to address strength anisotropy in cohesive soils, the commonly applied approach has been the anisotropy theory, which accounts for the rotation of the major principal stress axis, as introduced by Wei et al. (2024).

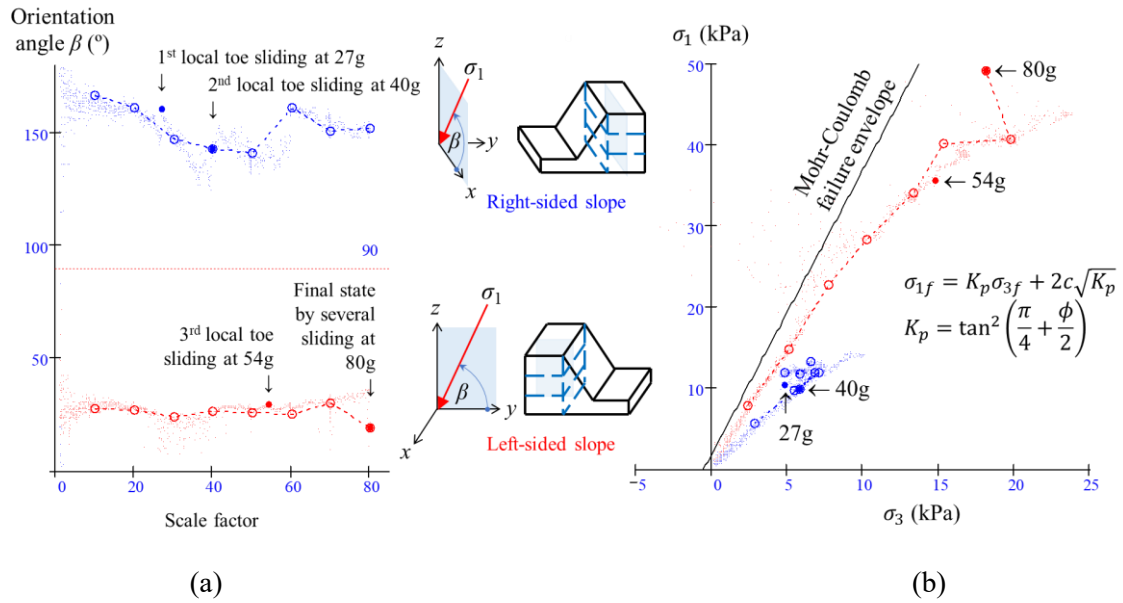


Fig. 4-49 The major principal stress' rotation and stress path in Case 1

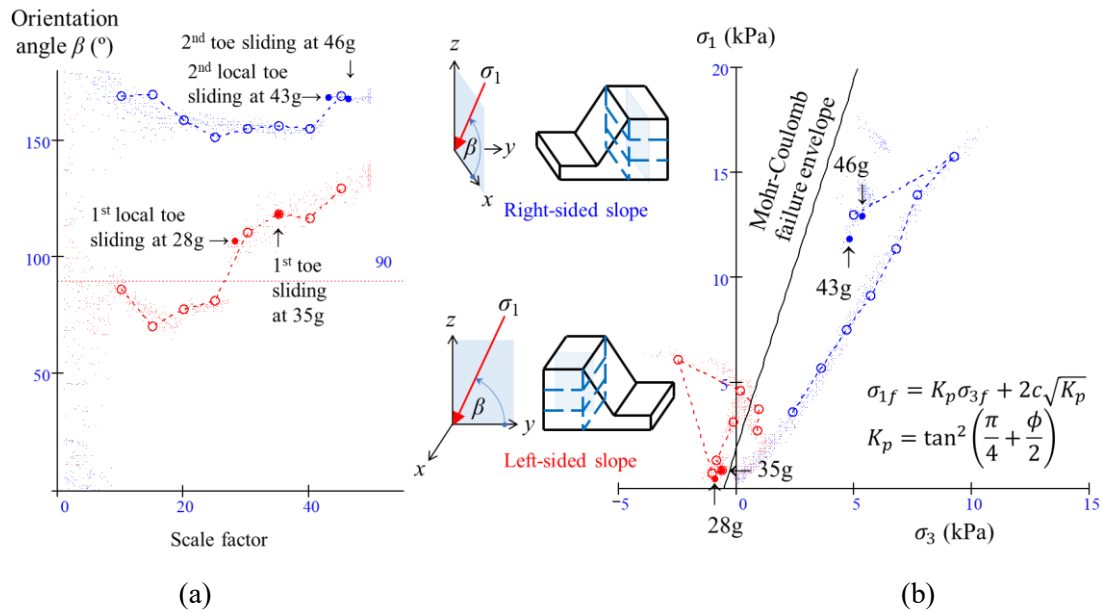


Fig. 4-50 The major principal stress' rotation and stress path in Case 3

The left **Fig. 4-49 (a)** shows variation in major principal stress of Case 1. The three-directional sensor

was oriented along the dip direction for the left slope. Initially, the major principal stress points nearly horizontally away from the slope surface, then shifts toward the slope surface, approximately 24° above the x -axis. This suggests that the slope is in a passive state around the sensor's location, which may explain the higher strength observed in the left slope's central part (see **Fig. 4-21**). The sensor was oriented along the strike direction for the right slope. The major principal stress was initially horizontal but gradually rotated clockwise with increasing centrifugal acceleration, then returns to horizontal immediately after the sliding happens. **Fig. 4-50 (a)** shows the variation in major principal stress for Case 3. In the left slope, the major principal stress remained vertical until the first sliding' onset, after which it rotated counterclockwise abruptly. For the right slope, the variation of the major principal stress' orientation follows a similar pattern to the observation in Case 1.

The stress paths in the principal stress space for the slope models of Case 1 and Case 3 are illustrated in **Fig. 4-49 (b)** and **Fig. 4-50 (b)**, respectively. In both cases, the stress path measured for the left slope in the slope direction was closer to the failure envelope than the path measured for the right slope in the strike direction from the beginning. This is because σ_3 in the strike direction represents the intermediate principal stress in the three-dimensional stress space and the Mohr–Coulomb circle constituted by the major and intermediate principal stress is less critical. For the left slope of Case 1, the stress path remained below the failure envelope and gradually deviated from it, corresponding to the progressive local toe sliding observed in this slope. In contrast, in the left slope of Case 3, the stress path crossed the failure envelope, aligning with the large-scale sliding observed in the experiment. Additionally, left slope in Case 1 and right slope in Case 3, which has smaller inclination, can reach higher σ_1 , indicating their greater stability than the other side.

4.5 Conclusions

The static stability of a laterally confined, uniform steep slope is examined in this chapter. Based on the 2D Culmann's method, a closed-form solution was proposed to avoid traditional iterative process. This method was then extended to 3D conditions by incorporating lateral resistance, using Rankine's theory. Centrifugal model tests on uniform slope models were conducted to validate the proposed theories. These findings are valuable not only for evaluating the steep slopes' stability at construction sites but also as a foundation for the following chapters on benched slopes and dynamic conditions. The main conclusions are listed as follows.

(1) The closed-form expression of the FS of the sliding wedge of a uniform slope was derived using Culmann's method, as shown in **Eq. (4-39)**. Unlike the original Culmann's method (**Eq. (4-8)**), this expression does not include the sliding angle (α), which in the original method is itself related to FS and cannot be determined directly. Additionally, the 2D sliding wedge method can be extended to 3D conditions by incorporating lateral resistance, which is related to the slope state and functions as an equivalent increase in internal cohesion.

(2) The observed failure mode for the uniform slope in the centrifugal tests under static loading was predominantly characterized by local toe sliding. Slope sliding typically did not occur along the entire crest at once but rather in partial, successive events as centrifugal acceleration increased. Sliding tended to initiate near the slope boundary, where the compaction degree was lower. With gradual mass loss, the central part of the slope consistently demonstrated the highest stability. Additionally, slopes with a 90% compaction degree exhibited significantly greater stability than those with 80% compaction, maintaining a steep inclination under high centrifugal loads, even without boundary support. As a result, slope sliding was primarily observed in slopes with an 80% compaction degree.

(3) Based on measurements from earth pressure sensors embedded in the slopes, internal stress distribution and variations of slope state were recorded. Sensors at the base of the model indicated that a steeper inclination induces greater stress concentration under the slope toe, thereby reducing slope stability. Measurements combining horizontal and vertical earth pressures showed that the slopes transitioned from a slightly passive to an active situation as centrifugal loading increased, which is reflected in the reduction factor (r_d). The r_d curve would rise abruptly following a sliding event, serving as an indicator of slope instability and the occurrence of sliding. Records of r_d demonstrated that the slope eventually reached a limit active state; therefore, 2D theory is adequate to analyze the uniform slope models' instability under static loading.

(4) When the mean shear strengths of Hiroshima sand, obtained from the DST, are used to predict the scaling factor relative to the slope angle, they form a lower boundary according to the closed-form solution. Given the variability in water content within the centrifugal models and the changes of shear strength with water content, the mean shear strengths with an upper 80% confidence interval were employed to account for discrepancies between predictions and experimental results. A satisfactory agreement was observed, validating the effectiveness of the proposed methods.

(5) The major principal stress' direction relied on the initial stress (compaction effect) and the

installation condition (stability of the surrounding soil). Changes can be observed during acceleration due to stress reorientation and redistribution, with some sudden changes if local failures occur. Progressive local toe sliding caused the stress path deviated from the failure envelope gradually. Large scale failure could happen when the stress path crossed the envelope.

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Chapter 5 Centrifugal modeling of benched slopes under static loading conditions

This chapter investigates benched slopes' stability under static loading through centrifugal model tests. The critical height and failure mode of benched slopes are examined. They are influenced by bench sizes and boundary conditions.

In **Section 5.1**, the closed-form solution of Culmann's method is applied under 2D conditions, derived in **Section 4.1.2**, is explained for benched slopes. Additionally, another potential sliding is evaluated by considering the first bench as a toe counterweight. The most potential sliding plane is recognized from the lowest FS for each failure mode.

In **Section 5.2**, two groups of centrifugal model tests on benched slopes are designed. In the first group, a first bench with varying dimensions is added to the original slope, which has an inclination of 65° . This group aims to study the impact of different first bench widths and heights on the second bench's stability, where buildings are typically situated. In the second group, the benched geometry is formed by excavating the original slope with an inclination of 70° . This group examines the impact of different excavation dimensions, while keeping the excavation weight constant, on the entire slope.

In **Section 5.3**, recorded centrifugal accelerations at the sliding's onset and observed instability modes of benched slopes are described. Additionally, data from installed sensors is presented to illustrate variations in earth pressure and displacement of the slopes during testing.

In **Section 5.4**, the experiments are compared with the 2D failure envelope. The state transitioned from passive to active in the benched slopes as centrifugal acceleration increases is also discussed.

Lastly, the conclusions obtained are listed in **Section 5.5**.

5.1 Theoretical consideration

5.1.1 Benched slope's failure mode

For a steep uniform slope as illustrated in **Fig. 5-1 (a)**, toe sliding on the sliding plane ② is considered the most likely failure mode. Here β represents the slope angle, α is the sliding surface inclination, and h means the slope height. Slope stability under this condition was analyzed in **Chapter 4**. For a simple benched slope with two benches, as shown in **Fig. 5-1 (b)**, three additional sliding surfaces (① ③ ④) can also exist.

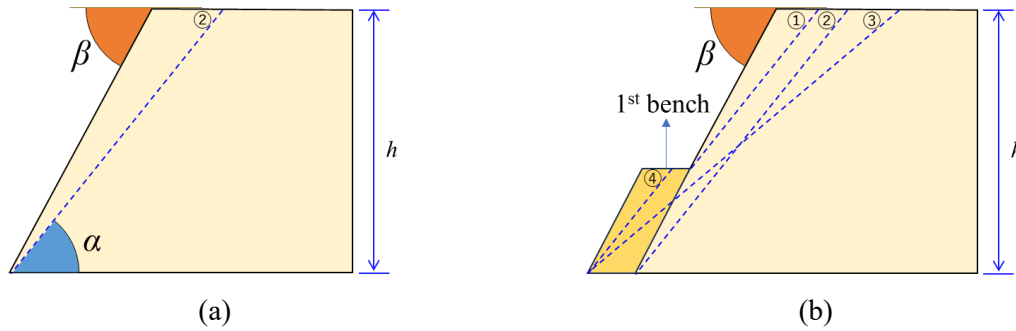


Fig. 5-1 Different failure modes due to slope geometry: (a) uniform slope; (b) benched slope

From the equation derived in **Section 4.1**, the *FS* for sliding modes ① and ④ can be calculated directly. For sliding mode ③, an equivalent slope must first be determined, as shown in **Fig. 5-2** (Tao et al., 2023). The benched slope can be viewed as the result of an excavation from an original uniform slope with the same slope angle. The parallelogram-shaped area, with a width of w_e and a height of h_e , represents the excavated part. According to the equivalent area in **Eq. (5-1)**, the excavated slope can be equivalent to a uniform slope with a smaller slope angle of $\tilde{\beta}$ as **Eq. (5-2)**. The equations previously derived for the uniform slope can then be applied to failure mode ③.

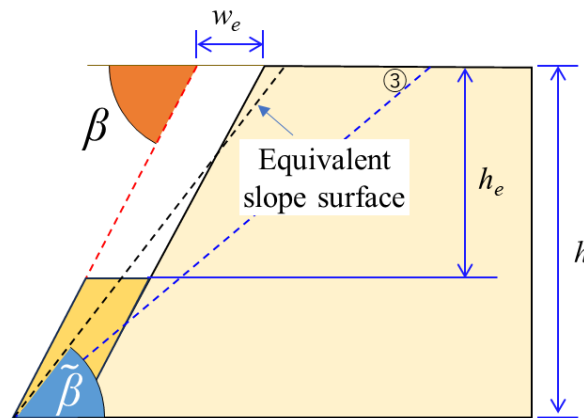


Fig. 5-2 Equivalent uniform slope to the benched slope

$$w_e h_e = \frac{1}{2} (\cot(\tilde{\beta}) - \cot(\beta)) h^2 \quad (5-1)$$

$$\tilde{\beta} = \text{atan}\left(\frac{1}{\frac{2w_e h_e}{h^2} + \cot(\beta)}\right) \quad (5-2)$$

However, the *FS* for the remained sliding mode ② cannot be determined immediately, as the resistance produced by the first bench must be considered.

5.1.2 Factor of safety of the second failure mode

As illustrated in **Fig. 5-3**, for the second failure mode, the first bench can be regarded as an external component at the toe of a uniform slope behind it, providing additional resisting force for the sliding section. Here, α is the failure plane inclination, β is the slope angle, h is the slope height, h_b is the first bench height, and b is the first bench width.

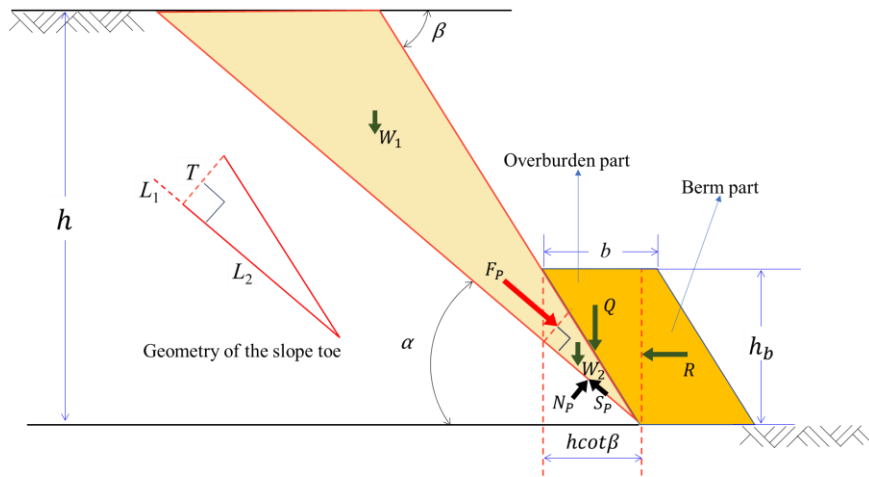


Fig. 5-3 Schematics of the second failure mode for the benched slope

$$L = h \csc(\alpha) \tag{5-3}$$

$$L_2 = h_b \cot(\beta) \sec(\alpha) \quad (5-4)$$

$$L_1 = L - L_2 \tag{5-5}$$

$$T = h_b \cot(\beta) \sec(\alpha) \tan(\beta - \alpha) \quad (5-6)$$

The failure plane's length is calculated by **Eq. (5-3)**. The sliding wedge is conceptually divided into a triangular toe section, with a weight of W_2 , and an upper section, with a weight of W_1 , separated by a virtual plane. First, a line is plotted vertically through the transition between the first bench and the upper slope, intersecting the sliding plane. From the intersection, a virtual plane is established, perpendicular to the sliding surface. Lengths of the toe wedge, L_2 , and the upper part, L_1 are expressed by **Eq. (5-4)** and **Eq. (5-5)**, respectively. The length of the virtual plane, T , is provided in **Eq. (5-6)**. This analysis focuses on the stability of the toe section. The entire sliding wedge remains stable if forces

acting on the toe reach equilibrium.

The entire sliding wedge's weight, W , the toe's weight, W_2 , and the upper section's weight, W_1 , can be calculated using Eq. (5-7), Eq. (5-8) and Eq. (5-9), respectively. Here, B denotes the model breadth, ρ_t denotes the total unit weight, n is the scaling factor in centrifuge modeling to account for gravity, and g represents the gravitation acceleration.

$$W = \frac{1}{2} \rho_t n g h^2 (\cot(\alpha) - \cot(\beta)) B \quad (5-7)$$

$$W_2 = \frac{1}{2} L_2 T B \rho_t n g \quad (5-8)$$

$$W_1 = W - W_2 \quad (5-9)$$

The unbalanced force from the upper part along the sliding surface, exerting on the virtual plane, F_P , is given by Eq. (5-13). F_{D1} is driving force in the upper part caused by its self-weight, S_{a1} represents the resisting force, which is generated by the normal force F_{N1} and cohesion, c , assuming the ground's shear strength is characterized by cohesion and friction angle, ϕ .

$$F_{D1} = W_1 \sin(\alpha) \quad (5-10)$$

$$F_{N1} = W_1 \cos(\alpha) \quad (5-11)$$

$$S_{a1} = c L_1 B + F_{N1} \tan(\phi) \quad (5-12)$$

$$F_P = F_{D1} - S_{a1} \quad (5-13)$$

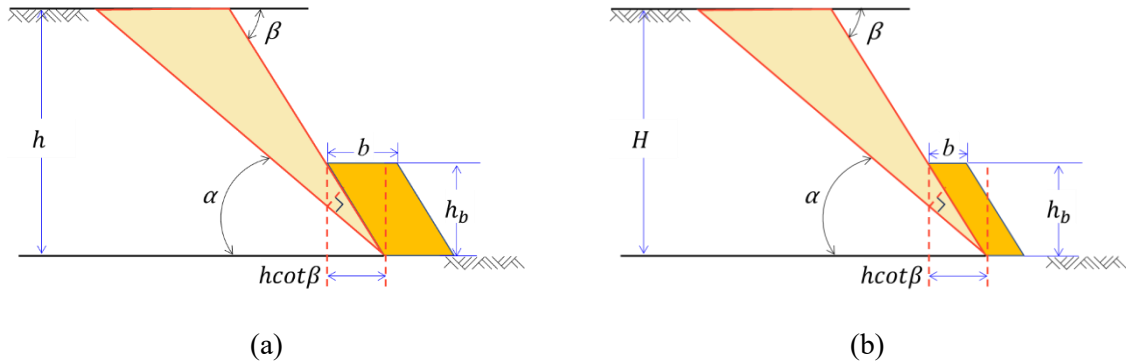


Fig. 5-4 Different width of the first bench compared to the width of the toe: (a) larger than the toe width; (b) smaller than the toe width

Pipatpongsa et al. (2023) noted that the stability of the slope toe plays a crucial role in the stability of the entire slope model situated on the bedding plane. Pariseau (2008) derived a simple solution for the factor of safety (FS) of a sliding wedge with a berm at its toe, where the berm provides additional resisting force by mobilizing shear strength at the berm bottom. In contrast, Pipatpongsa et al. (2024) used a different approach to account for the additional resistance provided by the toe counterweight, assuming that the overburden pressure exerted by the counterweight increases toe stability. Here, we

attempt to consider both methods as follows.

The resisting force from the first bench potentially consists of two components. Q represents the vertical overburden pressure provided by the volume of the first bench on the slope surface, and R is the horizontal resisting force generated by the remaining volume, acting as a berm (see **Fig. 5-3**).

When, $b \geq h \cot(\beta)$, as shown in **Fig. 5-4 (b)**, these two components can be calculated using **Eq. (5-14)** to **Eq. (5-16)**.

$$Q = \frac{1}{2} \rho_t n g h_b^2 \cot(\beta) B \quad (5-14)$$

$$W_R = \rho_t n g h_b B (b - \frac{1}{2} h_b \cot(\beta)) \quad (5-15)$$

$$R = c b B + W_R \tan(\phi) \quad (5-16)$$

When, $b < h \cot(\beta)$ as shown in **Fig. 5-4 (b)**, these two components can be calculated using **Eq. (5-17)** to **Eq. (5-19)**.

$$Q = \rho_t n g b (h_b - \frac{1}{2} b \tan(\beta)) B \quad (5-17)$$

$$W_R = \frac{1}{2} \rho_t n g b^2 \tan(\beta) B \quad (5-18)$$

$$R = c b B + W_R \tan(\phi) \quad (5-19)$$

Given the thrust force applied by the upper part, F_P , and additional resisting force produced by the first bench, Q and R , the total resisting force of the toe section, S_P and the driving force F_{D2} can be derived from **Eq. (5-20)** to **Eq. (5-22)**. Finally, the FS is calculated in **Eq. (5-23)**. Here, the presence of the berm section (R) restricts the downward movement of the overburden section (Q). As a result, Q is considered to only provide resisting force for the toe section.

$$N_P = (W_2 + Q) \cos(\alpha) + R \sin(\alpha) \quad (5-20)$$

$$S_P = c L_2 B + N_P \tan(\phi) + R \cos(\alpha) \quad (5-21)$$

$$F_{D2} = F_P + W_2 \sin(\alpha) \quad (5-22)$$

$$FS = \frac{S_P}{F_{D2}} \quad (5-23)$$

5.2 Experimental scheme

5.2.1 Test program

Two groups of static centrifugal models were prepared to examine static stability of slopes stabilized by benching. The centrifuge facility used for these tests is described in **Section 4.2**.

The first group includes four cases with eight model slopes, aimed at studying slopes with an additional bench section. The second group comprises five cases with ten model slopes, focused on examining benched slopes with equivalent excavation. For each case, two model slopes were tested simultaneously to improve efficiency and to facilitate the observation of the effects of different slope geometries. **Fig. 5-5** and **Fig. 5-6** illustrate benched slopes built before loading in the first and second groups, respectively. All model slopes were 18 cm in the total height and 15 cm in the slope height at model scale. Humid Hiroshima sand with $D_c=80\%$ and $w=8\%$ was used as the test material.

In the first group, two types of boundary conditions were applied by soil chambers with different widths, as described in **Section 4.2**. All benched slopes were divided into two stages, consisting of a first bench and a second bench (see **Fig. 5-5 (a)**). The first benches in this group had two different heights and widths (see **Fig. 5-5**). The height of the higher first bench was 7.5 cm on the model scale, half of the slope height (15 cm). The lower first bench's height was half of this, measuring 3.75 cm. In Case 3, the bench width was 6 cm on the model scale, which was twice the width of other cases (3 cm).

In the second group, the left slope with a slope angle of 70° in Case 1, as shown in **Fig. 5-6 (a)**, was considered as the original slope to be retrofitted. Apart from the original slope, all other slopes were subjected to equivalent excavation (with the excavated part's cross-section of 22.5 cm^2) to remain the same removed weight (see **Fig. 5-6**). The right slope in Case 1, having a reduced inclination of 60.58° , was a stabilized slope derived from the original slope by slope reduction. This specific reduced angle ensured that the first bench of the left slope in Case 2 maintained the same width and height as the higher bench in the first group. The inclination-reduced slope was tested again as the right slope in Case 2.

In Case 3, while the left benched slope from Case 2 was retained, the width and height of the left-hand side excavated area were interchanged for the excavated area on the right side. Case 4 repeated Case 2. In Case 5, the width and height of the excavation differed from those in Case 3.

Only the narrow soil chamber, as shown in **Fig. 4-8**, was used for the second group.

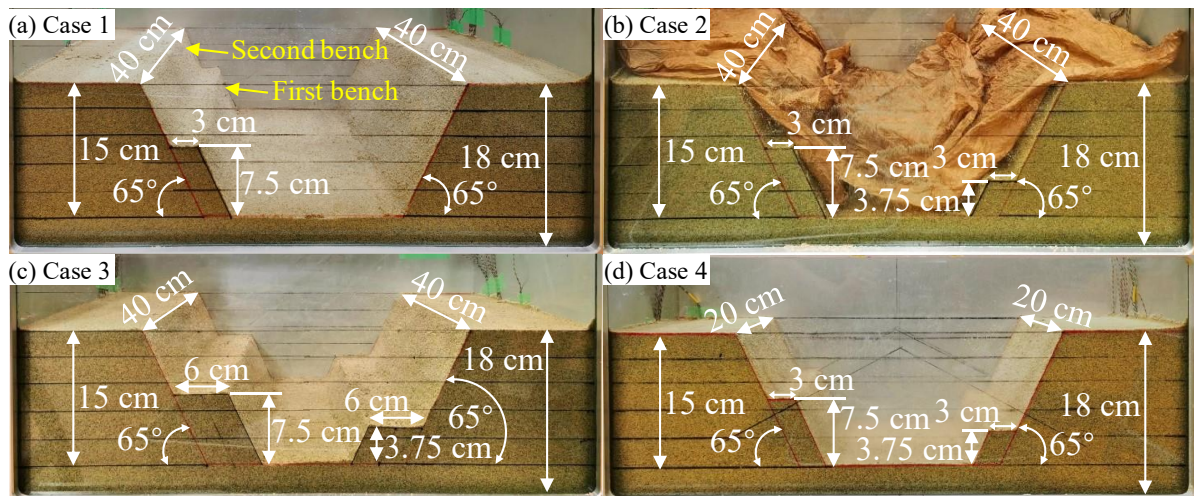


Fig. 5-5 Established model slopes before loading (model scale) in the first group

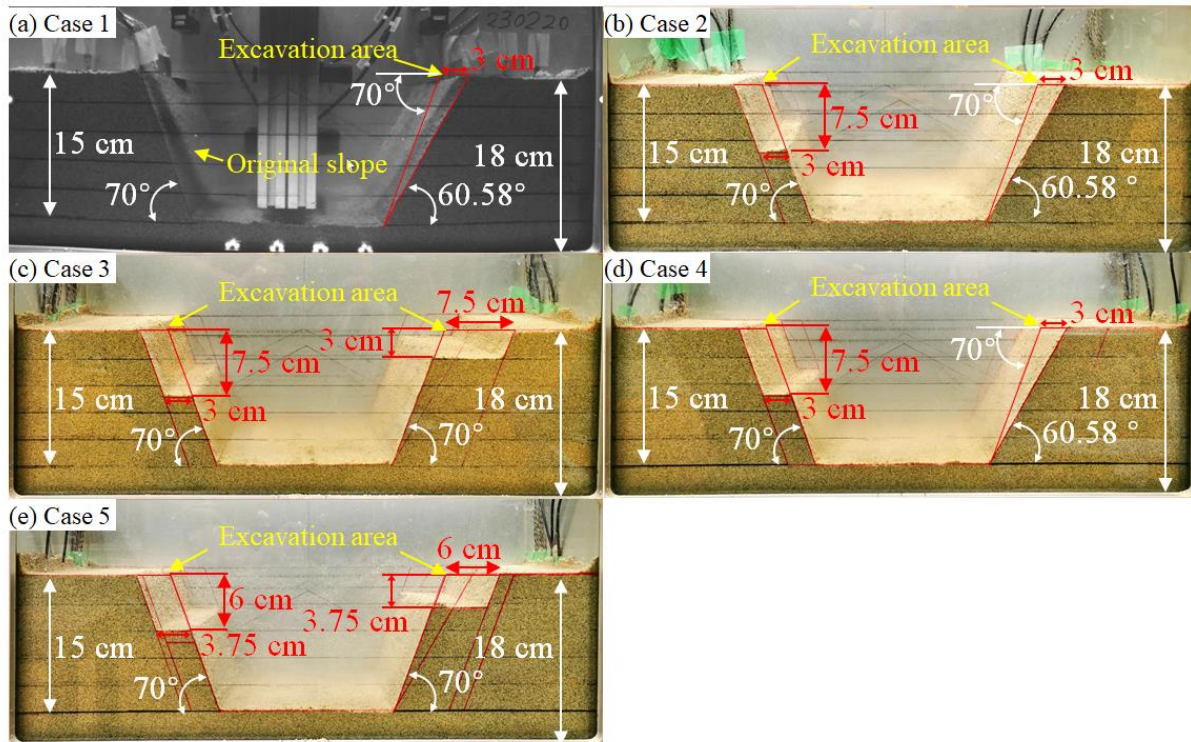


Fig. 5-6 Established model slopes before loading (model scale) in the second group

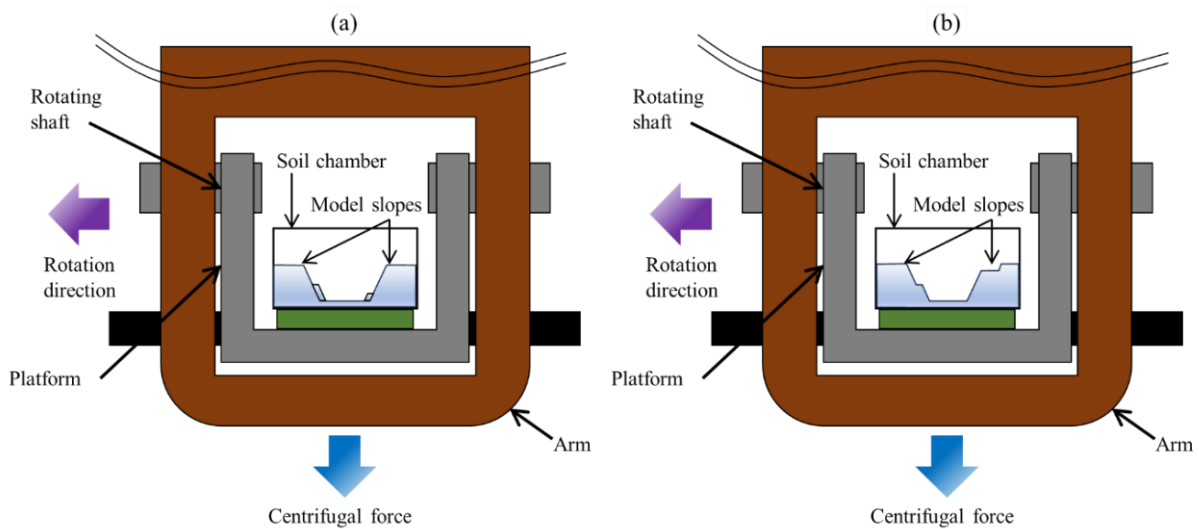
Table 5-1 provides the detailed model configurations for all four cases in the first group, including the case number, experimental date, type of soil chamber used, degree of compaction (D_c), slope angle (β), height of the first bench (B_H), width of the first bench (B_W), initial water content (w), and the loading process. A maximum centrifugal loading of 90G was applied in these tests to trigger slope sliding. **Table 5-2** presents the information for the second group, where the maximum centrifugal loading was 100G. In this group, E_H denotes the height of the excavated part, and E_W represents its width. A diagram illustrating model slopes under centrifugal loading for both groups is shown in **Fig. 5-7**.

Table 5-1 Model configurations for slopes with additional bench height under static loading

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)	1 st bench height B_H (cm)		1 st bench width B_W (cm)		Initial water content, w (%)	Loading process (G)
					Left	Right	Left	Right		
1	220706	Wide	80	65	7.5	3.75	3	—	8	1 → 50 → 60 → 70 → 80 → 90 → 1
2	220801	Wide					3	—		
3	220803	Wide					6	—		
4	220812	Narrow					3	—		

Table 5-2 Model configurations for benched slopes with equivalent excavation under static loading

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)		Height of excavation, E_H (cm)		Width of excavation, E_W (cm)		Initial water content, w (%)	Loading process (G)
				Left	Right	Left	Right	Left	Right		
1	230220	Narrow	80	70	60.58	—		3		8	1→50→1
2	230224			70	60.58	7.5	—	3	3		1→50→60→70→80→90→100→50→1
3	230227			70	70	7.5	3	3	7.5		1→50→60→70→80→90→1
4	230301			70	60.58	7.5	—	3	3		1→50→60→70→80→90→100→50→1
5	230303			70	70	6	3.75	3.75	6		1→50→60→66→1
											1→70→80→90→100→1

**Fig. 5-7** Diagram of model slopes during centrifugal loading: (a) the first group; (b) the second group

5.2.2 Instrumentation

The specifications of the earth pressure gauges, laser sensors, and other sensors employed in this chapter are identical to those described in **Section 4.2.2**.

A maximum of 29 earth pressure gauges were placed within the model to measure internal earth pressure. Additionally, up to six laser sensors were installed to record slope displacements at specific locations during testing. Some earth pressure gauges were mounted on rigid triangular bases, forming three-directional earth pressure gauges

The sensor arrangement varied slightly in each case. Most sensors were positioned at the bottom, one-third height, and two-thirds height of the model (see **Fig. 5-8** to **Fig. 5-12**). Eight earth pressure sensors (P6, P7, P8, P9, P12, P13, P14, P15) were placed on the slope boundary to monitor horizontal earth pressures, while nine additional sensors (P1, P2, P3, P4, P5, P10, P11, P16, P17) measured vertical

earth pressures. In Cases 2 through 5 of the second group (see **Fig. 5-13** and **Fig. 5-14**), six additional sensors were installed at the interface between the first bench and the slope (P24, P25, P26, P27, P28, P29).

The model slopes' displacement was also measured by laser sensors. The first three cases of the first group installed two laser sensors (LD1 and LD4) to measure the displacement of both slope crests in the vertical direction. In the last case of the first group, two additional laser sensors (LD2 and LD3) were added to record the slope crests horizontal displacement. The second group installed two additional sensors (LD3 and LD4) to monitor displacement in the horizontal direction at the mid-height of the slope surfaces. Targets, introduced in **Section 4.2.2**, were also placed.

In all cases, sensors P18, P19, P20, P21, P22, and P23 were combined to form two three-directional earth pressure gauges. These gauges were placed at two-thirds of the model height and positioned between the longitudinal centerline and the back wall of the soil chamber.

In the second group, six moisture sensors were used to measure variations in water content within the slopes. These sensors were positioned at the bottom, middle, and top of the slopes. Specific depths of the moisture sensors in Case 1 differ slightly from those in other cases (see **Fig. 5-12**, **Fig. 5-13** and **Fig. 5-14**). Additionally, high-speed cameras were employed in each case to monitor slope deformation, providing visual evidence to complement the sensor measurements. The sensor configurations for all cases are listed in **Table 5-3**, **Table 5-4** and **Table 5-5**.

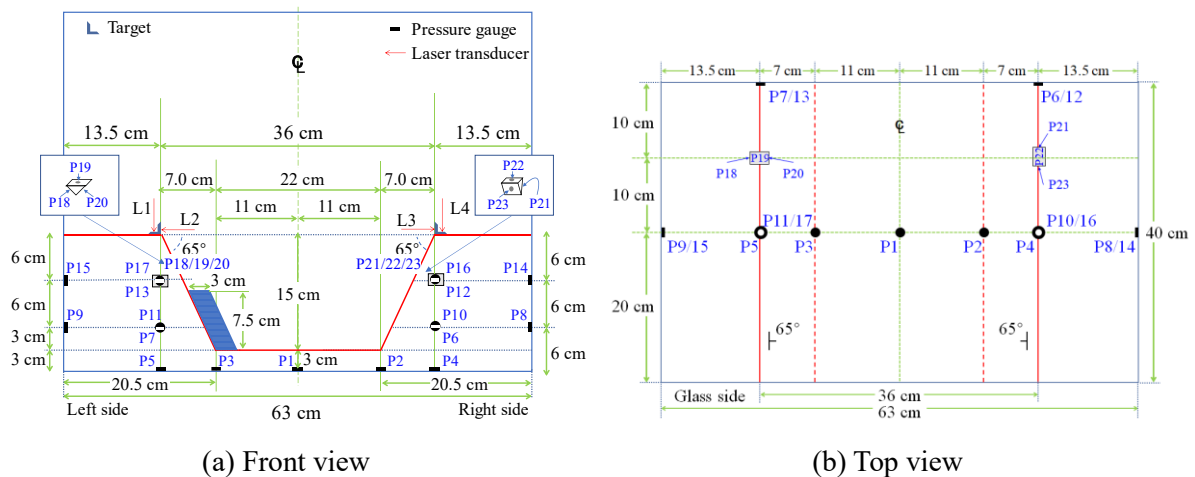
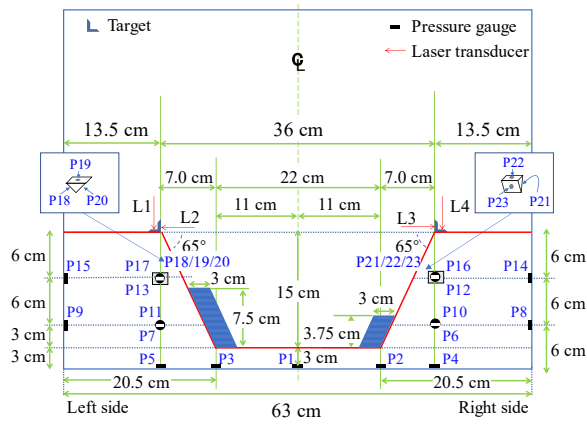
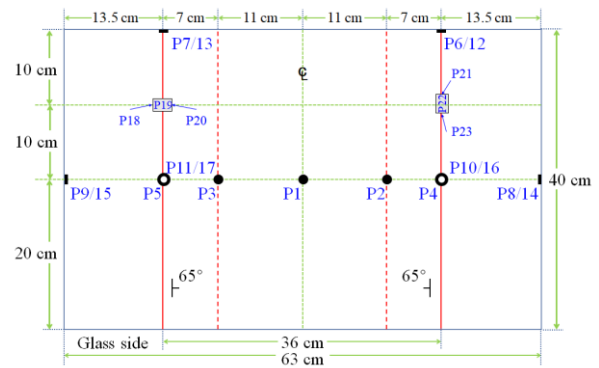


Fig. 5-8 Configuration of sensors in Case 1 in the first group

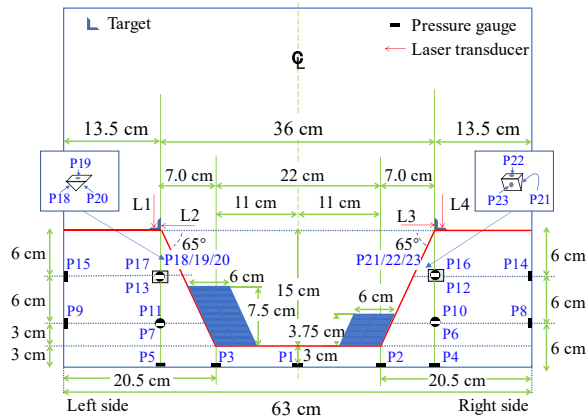


(a) Front view

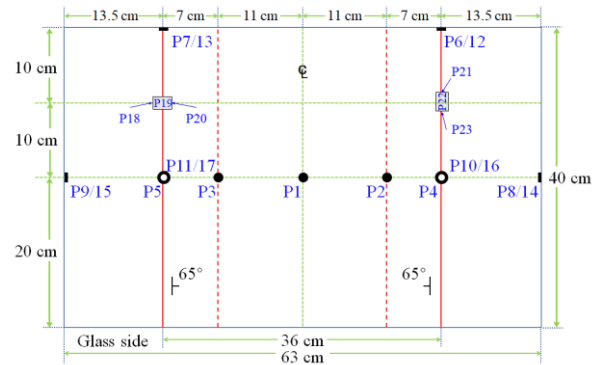


(b) Top view

Fig. 5-9 Configuration of sensors in Case 2 in the first group

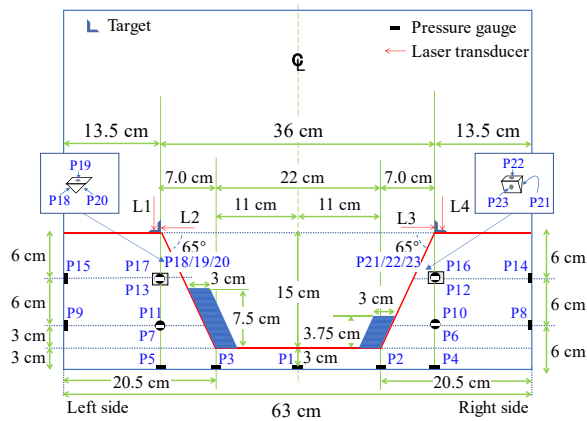


(a) Front view

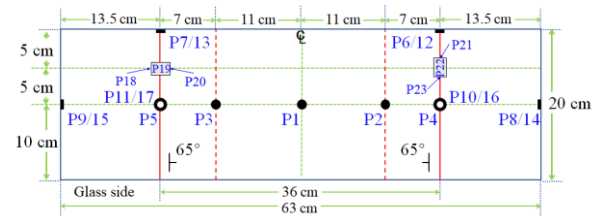


(b) Top view

Fig. 5-10 Configuration of sensors in Case 3 in the first group

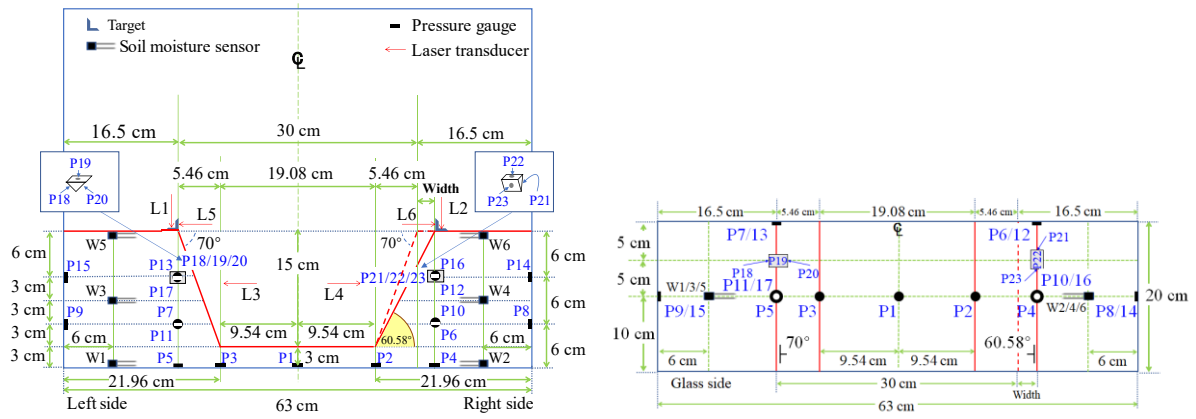


(a) Front view



(b) Top view

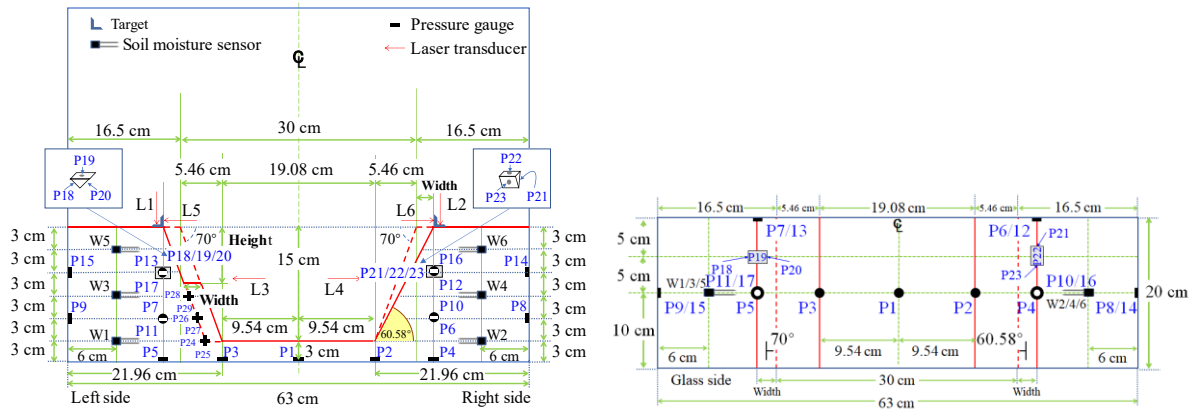
Fig. 5-11 Configuration of sensors in Case 4 in the first group



(a) Front view

(b) Top view

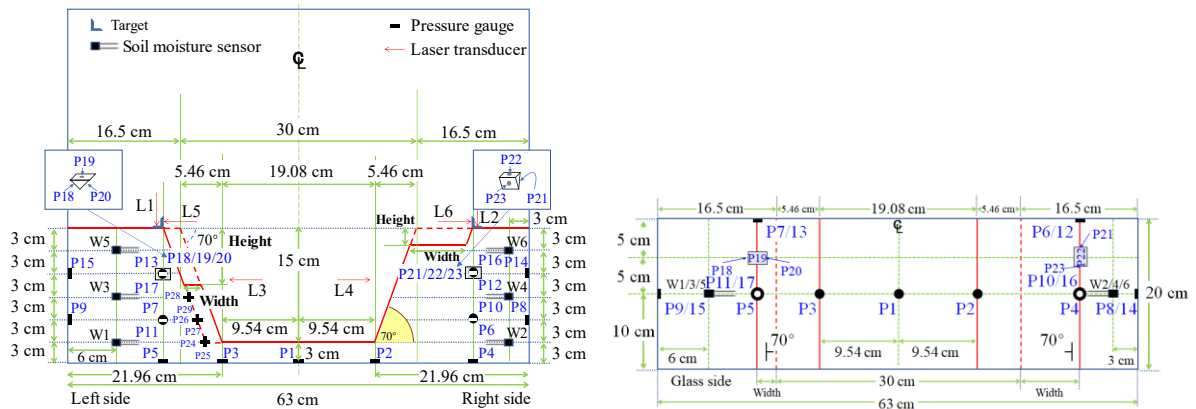
Fig. 5-12 Configuration of sensors in Case 1 in the second group



(a) Front view

(b) Top view

Fig. 5-13 Configuration of sensors in Case 2 and Case 4 in the second group



(a) Front view

(b) Top view

Fig. 5-14 Configuration of sensors in Case 3 and Case 5 in the second group

Table 5-3 List of sensors for all cases in the first group

No.	Date	Earth pressure sensor																							Laser sensor			
		P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	LD	LD	LD	LD
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	1	2	3	4
1	220706	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	-	-	○
2	220801	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	△	○	○	○	○	○	○	○	-	-	○
3	220803	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	-	-	○
4	220812	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	△

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned

- The sensor was not placed

Table 5-4 List of earth pressure sensors for all cases in the second group

No.	Date	Earth pressure sensor																													
		P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14	P15	P16	P17	P18	P19	P20	P21	P22	P23	P24	P25	P26	P27	P28	P29	
1	230220	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	-					
2	230224	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
3	230227	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
4	230301	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
5	230303	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned

- The sensor was not placed

Table 5-5 List of other sensors in the second group

No.	Date	Laser sensor						Moisture sensor					
		LD	LD	LD	LD	LD	LD	W	W	W	W	W	W
		1	2	3	4	5	6	1	2	3	4	5	6
1	230220	○	○	○	○	○	○	○	○	○	○	○	○
2	230224	○	○	○	○	○	○	○	○	○	○	○	○
3	230227	○	○	○	○	○	○	○	○	○	○	○	○
4	230301	○	○	○	○	○	○	○	○	○	○	○	○
5	230303	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned

- The sensor was not placed

5.2.3 Test procedure

The experimental process of each case was similar. First, dry Hiroshima sand was blended with water to achieve a desired water content of 8%. Prepared sand was then divided into six equal portions and

compacted into the soil chamber in six layers using a wooden hammer, with each layer being 3 cm thick. Each layer's weight was calculated from the degree of compaction ($D_c=80\%$) and the volume of each layer. Sensors were placed as the soil was compacted to their respective heights. After forming a horizontal foundation with a height of 18 cm, unnecessary soil was excavated from the center, and two slope surfaces were shaped using a spatula. The preparation process is illustrated in **Fig. 5-15**.

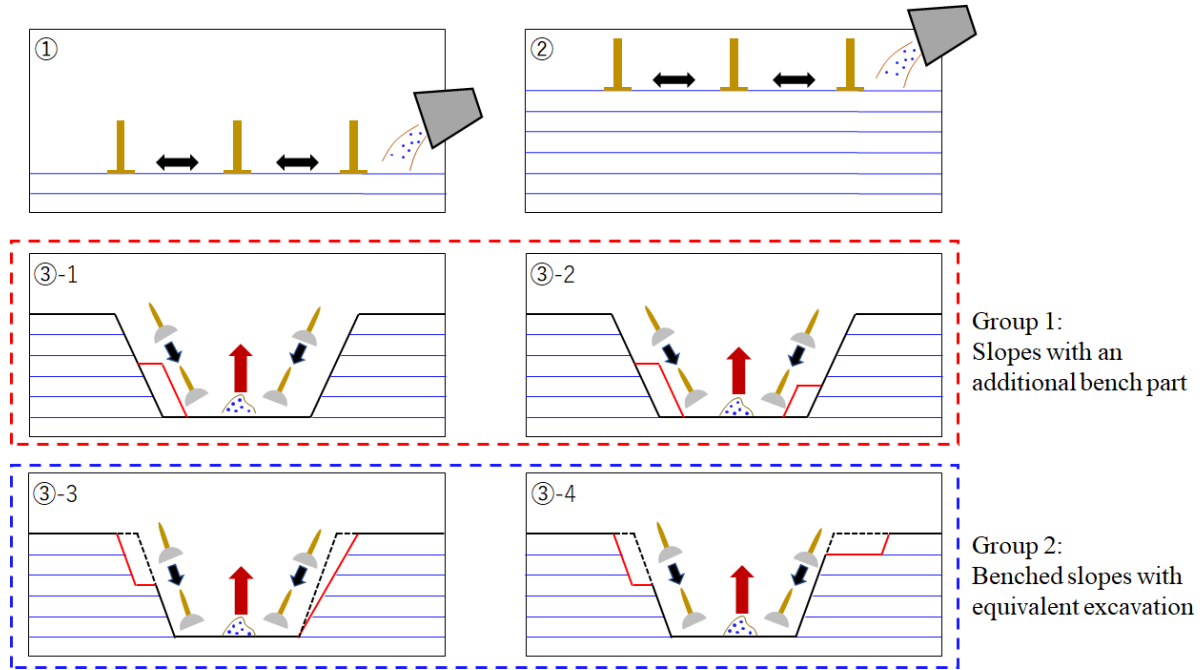


Fig. 5-15 Flow chart of the construction procedure for the model slope

In Group 1, the bench slope was constructed by adding a bench section to the original slope, while in Group 2, the bench slope was created by excavating the original slope. The uniform slopes shown in ③-1 and ③-3 were prepared for comparison with the bench slope.

The model was then installed on the centrifuge platform for testing. Centrifugal loading was initially increased steadily from 1G to 50G, followed by increments of 10G for safety and recording convenience. Once the centrifuge reached maximum loading, it was stopped, and the centrifugal acceleration was gradually decreased back to 1G. In Cases 2 and 4 of the second group, the centrifuge was held at 50G for a period during the unloading stage due to specific testing procedures. In Case 5 of the second group, an incident occurred during the first loading, resulting in the centrifuge being stopped at 66G and then restarted.

The detailed loading process for each case is listed in **Table 5-1** and **Table 5-2**.

5.3 Experimental results

During all cases, details of the slope sliding process were captured by the high-speed camera. All sliding events occurred within one second, which will be shown in the time order in the section for each case. Additionally, the stress and displacement measured by installed sensors are presented as follows.

5.3.1 Laterally confined slopes with an additional bench part

5.3.1.1 Case 1 ($\beta=65^\circ$, $B_{HL}=7.5$ cm, $B_{WL}=3$ cm, model breath =40 cm)

Sliding process

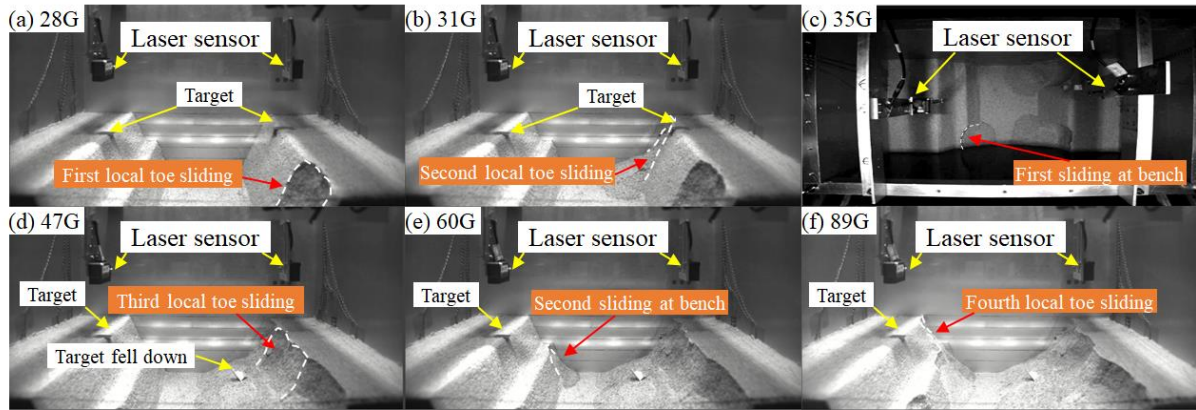


Fig. 5-16 Sliding process in Case 1 of the first group: (a) first local toe sliding on the right slope; (b) second local toe sliding on the right slope; (c) first sliding at bench on the left slope; (d) third local toe sliding on the right slope; (e) second sliding at bench on the left slope; (f) fourth local toe sliding on the left slope

The definitions of different types of sliding are clarified as follows:

Toe Sliding: The sliding surface connects the slope crest and the toe across the entire slope breath.

Local Toe Sliding: The sliding surface connects the slope crest and the toe but across only partial breath.

Bench Sliding: The sliding area is confined to the bench level.

In Case 1 of the first group, only the left slope was constructed with an external bench, with a height equal to half of the slope height and a width of 3 cm at model scale. The right-hand side was a uniform slope without toe protection, making it noticeably weaker. A local toe sliding first occurred near the glass side at 28G (see **Fig. 5-16 (a)**), followed by a local toe sliding near the back wall at 31G (see **Fig. 5-16 (b)**). The first bench sliding on the left slope initiated on the glass side at 35G (see **Fig. 5-16 (c)**). Due to camera limitations, this sliding is not visible from the front view, so the top view is presented.

The right slope's failed area expanded with increasing load, and the remaining central portion failed at 47G (see **Fig. 5-16 (d)**), forming a complete sliding surface. For the left slope, another sliding event at the first bench occurred near the back wall at 60G (see **Fig. 5-16 (e)**), followed by several additional slides at the first bench. However, even up to the maximum acceleration of 89G, only a small local toe sliding occurred on the left slope from crest to toe (see **Fig. 5-16 (f)**), indicating the protective capability of the first bench for the second bench.

Displacement

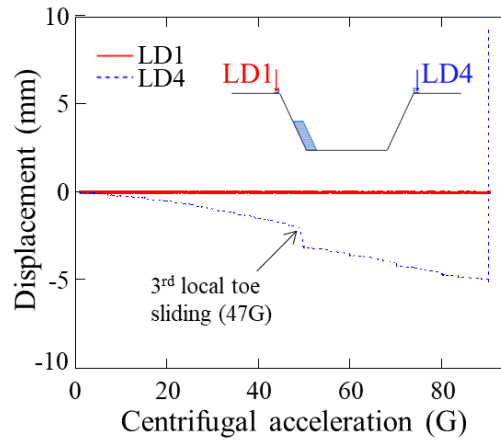


Fig. 5-17 Vertical displacement of slope crests in Case 1 of the first group with increasing centrifugal acceleration

In Case 1 of the first group, displacement of the left crest in the vertical direction (LD1) remained zero throughout the test, indicating the stability of the second bench on the left side (see **Fig. 5-17**). In contrast, the crest on the right side settled gradually as centrifugal acceleration increased, with LD4 showing a gradual decrease. A small jump in LD4 occurred at 47G due to the target falling during the third local toe sliding on the right slope.

Earth pressure

Fig. 5-18 presents the correlation of non-dimensional earth pressures with centrifugal acceleration at three different heights of Case 1. The earth pressure (P) is normalized by $\rho g H$ (ρ : total density of the sand, g : gravitational acceleration, H : model height). These symbol definitions remain consistent in the following cases. All sensors show an overall increasing trend with occasional drops, indicating stress release caused by sliding. Pressure sensor P1, which measures vertical earth pressure at the central part of the model bottom, records the smallest values among all bottom sensors (**Fig. 5-18 (a)**). Additionally, the values from the sensors on the three-directional gauge in the right slope fluctuate more significantly than those in the left slope (**Fig. 5-18 (d) and (e)**).

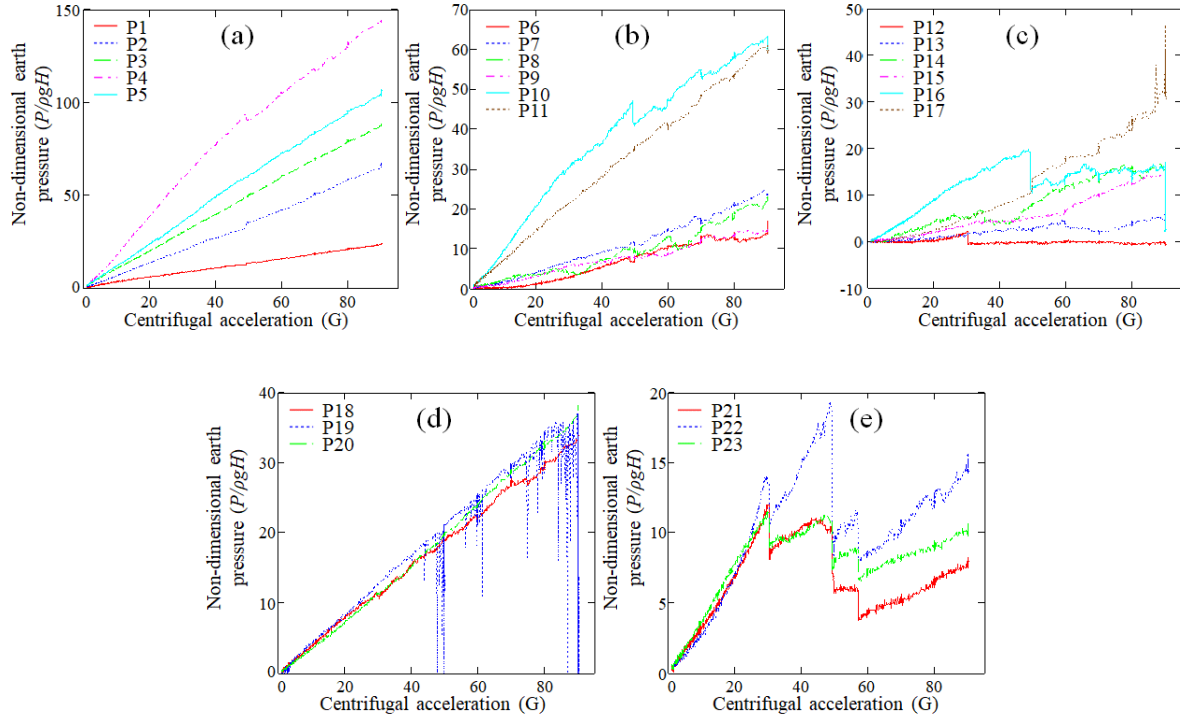


Fig. 5-18 Measured earth pressure with increasing centrifugal acceleration in Case 1 of the first group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

5.3.1.2 Case 2 ($\beta=65^\circ$, $B_{HL}=7.5$ cm, $B_{WL}=3$ cm, $B_{HL}=3.75$ cm, $B_{WL}=3$ cm, model breath =40 cm)

Sliding process

In Case 2, the shape of the left slope remained the same as in Case 1, while a lower first bench with half the height of the left one (3.75 cm) was constructed for the right slope. The bench width on both sides remained 3 cm. The first sliding occurred at the left first bench near the glass side at 66G (see **Fig. 5-19 (a)**), followed by another bench sliding on the left slope at 68G near the back wall (see **Fig. 5-19 (b)**). The first local toe sliding on the right slope took place at 69G near the glass side (see **Fig. 5-19 (c)**), expanding along the crest at 78G (see **Fig. 5-19 (d)**). The presence of the lower first bench on the right slope delayed failure compared to the right uniform slope in Case 1. As in the previous case, no failure occurred on the second bench above the first bench on the left slope.

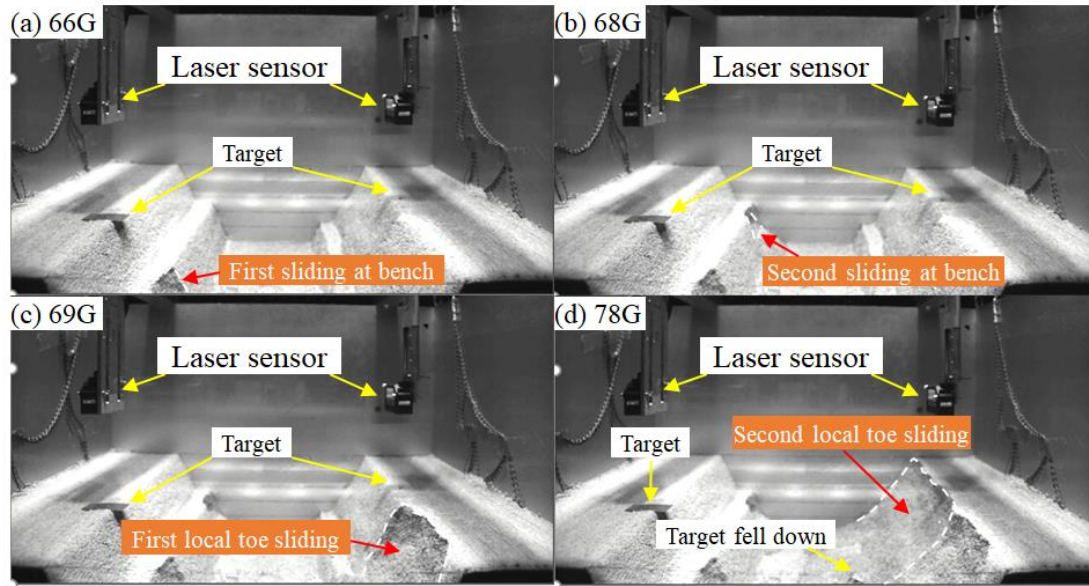


Fig. 5-19 Sliding process in Case 2 of the first group: (a) first sliding at bench on the left slope; (b) second sliding at bench of the left slope; (c) first local toe sliding on the right slope; (d) second local toe sliding on the right slope

Displacement

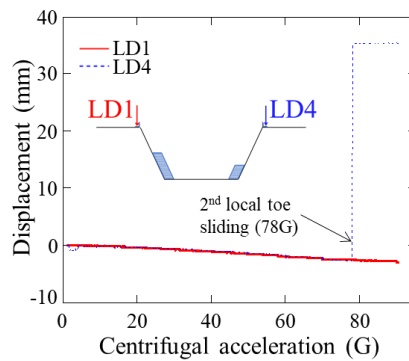


Fig. 5-20 Slope crests' vertical and horizontal displacement of Case 2 of the first group with increasing centrifugal acceleration

In this case, the settlement magnitudes (LD1 and LD4) on both slopes were similar, decreasing steadily with increasing centrifugal acceleration. LD4 went out of range at 78G due to the target falling on the right slope, which was caused by the second local toe sliding (see **Fig. 5-20**).

Earth pressure

Fig. 5-21 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 2 of the first group. The value of P19 becomes unstable after 60G because of malfunction.

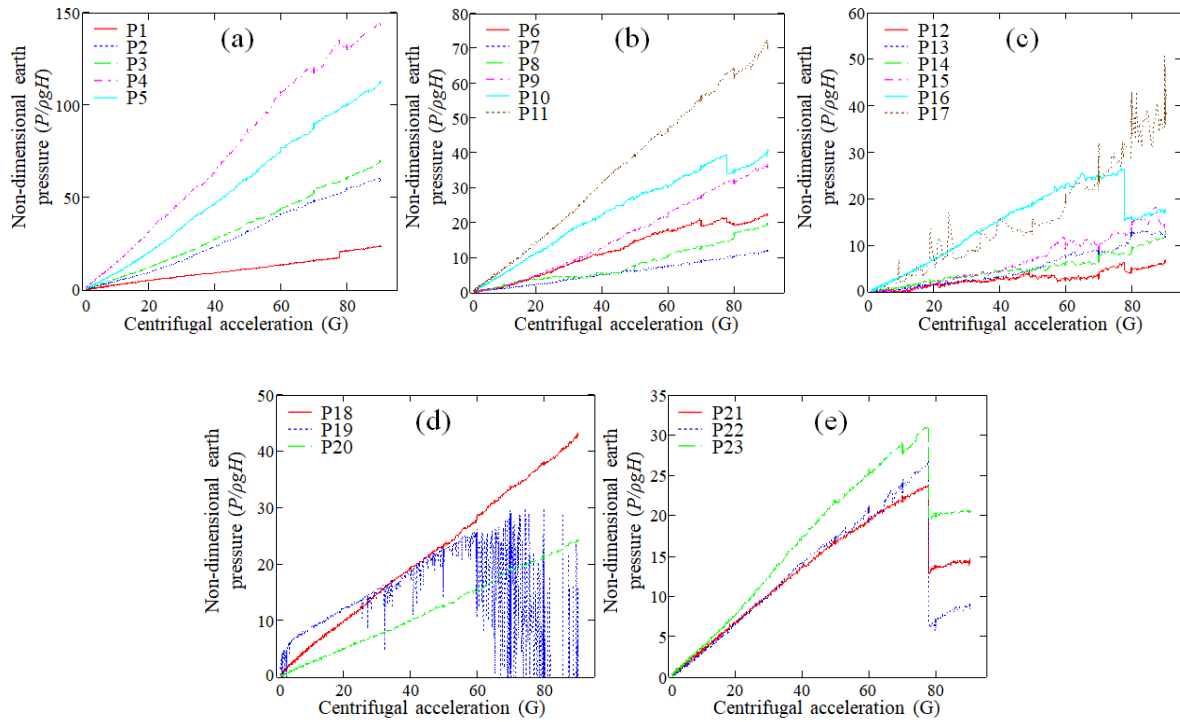


Fig. 5-21 Measured earth pressure with increasing centrifugal acceleration in Case 2 of the first group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

5.3.1.3 Case 3 ($\beta=65^\circ$, $B_{HL}=7.5$ cm, $B_{WL}=6$ cm, $B_{HL}=3.75$ cm, $B_{WL}=6$ cm, model breath =40 cm)

Sliding process

In Case 3, the first bench width on both sides was doubled compared to Case 2. The first sliding occurred on the left first bench at 47G (see **Fig. 5-22 (a)**). The second sliding event occurred on the right slope's second bench near the back wall at 55G (see **Fig. 5-22 (b)**). The sliding area on the left first bench continued to expand, and by 67G, it spanned the model's entire breadth (see **Fig. 5-22 (c)**). At 70G, a sliding event appeared on the left slope's second bench near the glass (see **Fig. 5-22 (d)**). Next, a bench sliding occurred on the right slope near the glass side at 76G (see **Fig. 5-22 (e)**). The final bench sliding took place on the right slope's second bench near the back wall at 89G (see **Fig. 5-22 (f)**).

The test ended with the major part of the left slope's second bench and the right slope's central area remaining stable. This outcome indicates that a wider first bench provides greater support to the second bench, transforming the failure mode from a combination of toe sliding and bench sliding to primarily bench sliding.

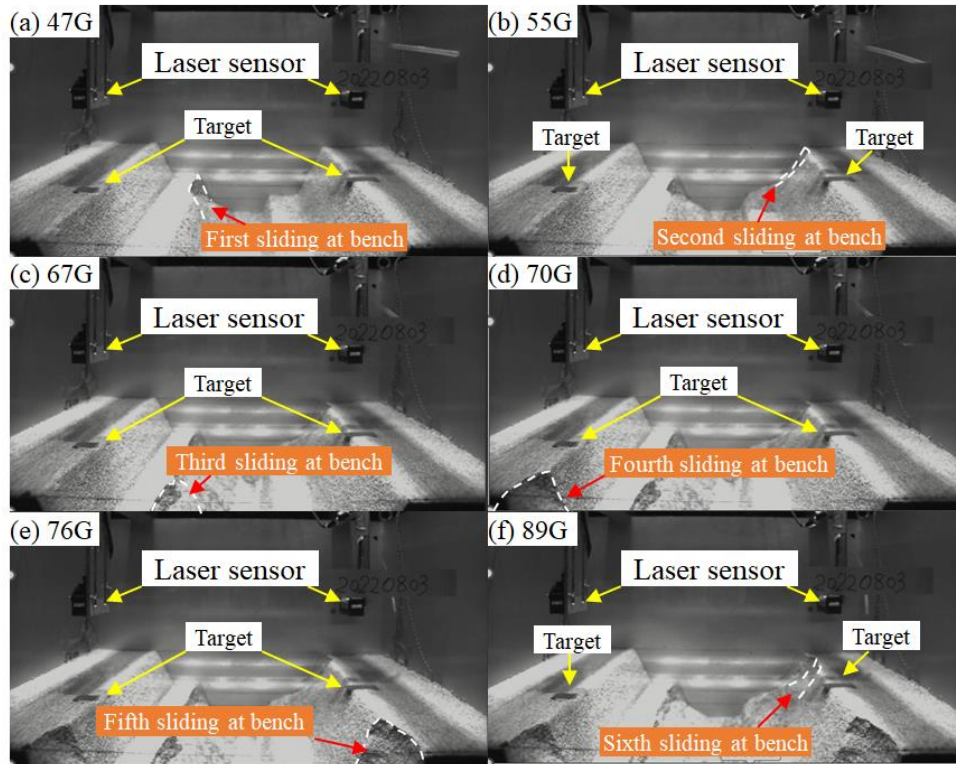


Fig. 5-22 Sliding process in Case 3 of the first group: (a) first sliding at bench on the left slope; (b) second sliding at bench on the right slope; (c) third sliding at bench on the left slope; (d) fourth sliding at bench on the left slope; (e) fifth sliding at bench on the right slope; (f) sixth sliding at bench on the right slope

Displacement

In this case, the settlement magnitudes (LD1 and LD4) on both slopes are similar, with values decreasing steadily as centrifugal acceleration increases (see **Fig. 5-23**). The value of LD4 is smaller than that of LD1, indicating greater deformation of the right slope with a lower first bench.

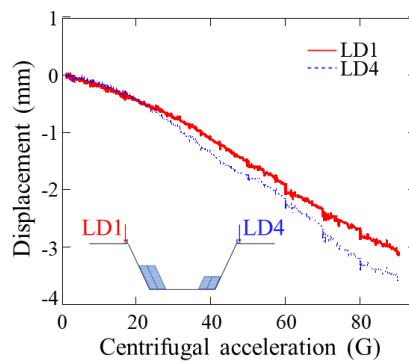


Fig. 5-23 Slope crests' vertical and horizontal displacement of Case 3 of the first group with increasing centrifugal acceleration

Earth pressure

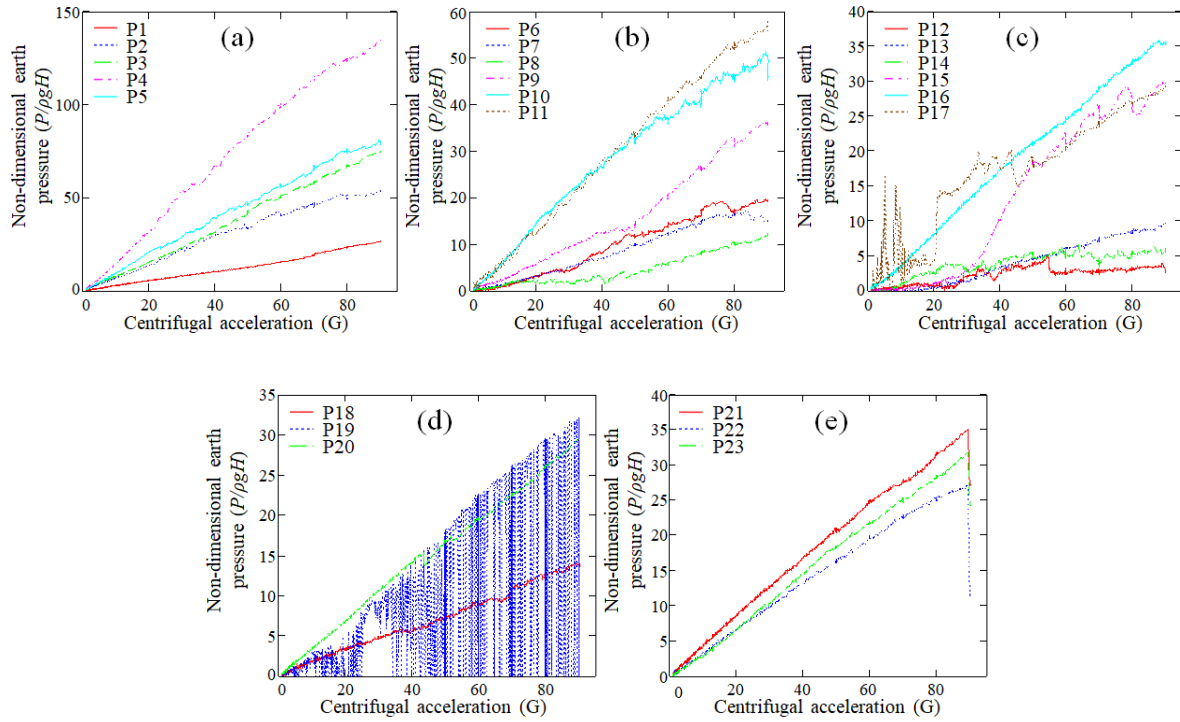


Fig. 5-24 Measured earth pressure with increasing centrifugal acceleration in Case 3 of the first group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

Fig. 5-24 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 3 of the first group. Similar to Case 2, the value of P19 became unstable during loading because of malfunction. The three-directional gauge in the right slope recorded more stable values in Case 3 than in Case 2, suggesting that a wider bench width enhances slope stability.

5.3.1.4 Case 4 ($\beta=65^\circ$, $B_{HL}=7.5$ cm, $B_{WL}=3$ cm, $B_{HL}=3.75$ cm, $B_{WL}=3$ cm, model breath =20 cm)

Sliding process

In the final case of the first group, the narrow soil chamber was used to test the boundary effect. The primary difference from Case 2 is the model breadth. The right slope's first local toe sliding occurred around 58G (see **Fig. 5-25 (a)**), followed by the left first bench's sliding near the glass side at 59G, which extended across the model width by 80G (see **Fig. 5-25 (b)** and **(c)**). For the right slope, the final local toe sliding occurred at 82G, forming a complete sliding surface (see **Fig. 5-25 (d)**). Compared to Case 2, the final local toe sliding on the right slope was delayed, indicating greater stability in the slope

with a narrower boundary condition.

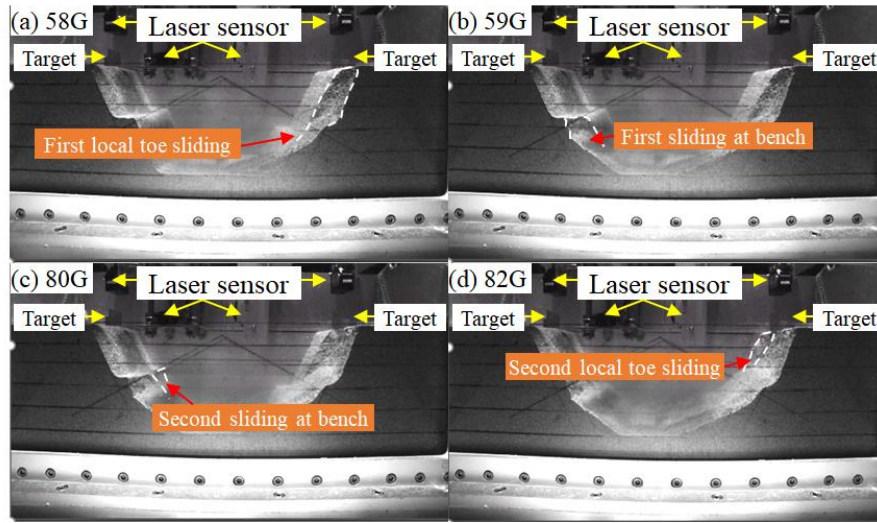


Fig. 5-25 Sliding process in Case 4 of the first group: (a) first local toe sliding on the right slope; (b) first sliding at bench on the left slope; (c) second sliding at bench on the left slope; (d) second local toe sliding on the right slope

Displacement

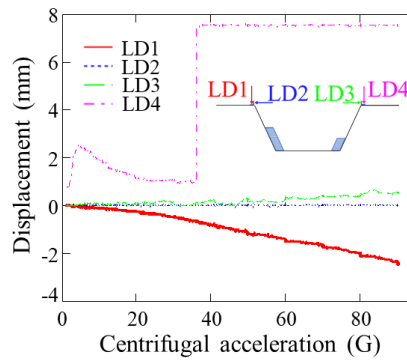


Fig. 5-26 Slope crests' vertical and horizontal displacement of Case 4 of the first group with increasing centrifugal acceleration

In this case, LD4 malfunctioned (see **Fig. 5-26**). Based on the values of LD2 and LD3, the slope crests shifted toward the center of the model, as indicated by a slight increase in their measurement.

Earth pressure

Fig. 5-27 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 4 of the first group.

The experimental results for all cases of the first group are listed in **Table 5-6**. After the testing, the

water content in the models was sampled from three distinct heights from the bottom of each model.

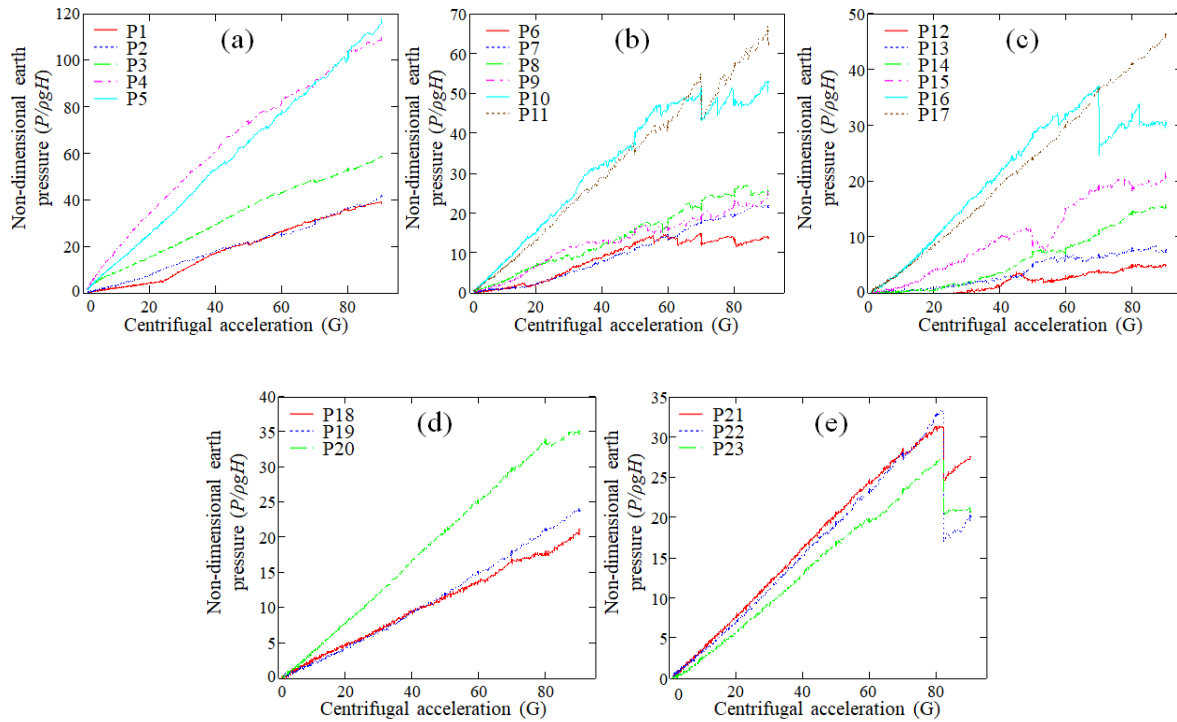


Fig. 5-27 Measured earth pressure with increasing centrifugal acceleration in Case 4 of the first group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

Table 5-6 The experimental results of the first group of tests

No.	Date	Final water content (%)			Sliding G			
		Bottom	Middle	Top	Left		Right	
		(0-2 cm)	(8-10 cm)	(16-18 cm)	First	Last	First	Last
1	220706	8.7	6.9	5.7	60G	89G	28G	47G
2	220801	9.1	7.3	5.2	66G	68G	69G	78G
3	220803	8	7.5	6.2	47G	70G	55G	89G
4	220812	9.2	7.3	5.5	59G	80G	58G	82G

5.3.2 Laterally confined benched slopes with equivalent excavation

5.3.2.1 Case 1 ($\beta_L=70^\circ$, $\beta_R=60.58^\circ$, $E_{HL}=0$ cm, $E_{WL}=0$ cm, $E_{HR}=0$ cm, $E_{WR}=3$ cm)

Sliding process

In Case 1 of the second group, the right slope was modified by reducing its inclination from 70° to 60.58° , while the left side remained as the original slope. The right slope, with a gentler inclination, was

noticeably stronger than the left side. At 28G, a local toe sliding occurred near the back wall on the original slope (see **Fig. 5-28 (a)**), and a complete toe sliding surface formed following another local toe sliding at 35G (see **Fig. 5-28 (b)**). The right slope remained stable until 43G, when a local toe sliding took place near the back wall (see **Fig. 5-28 (c)**), with a complete sliding surface appearing at 46G (see **Fig. 5-28 (d)**).

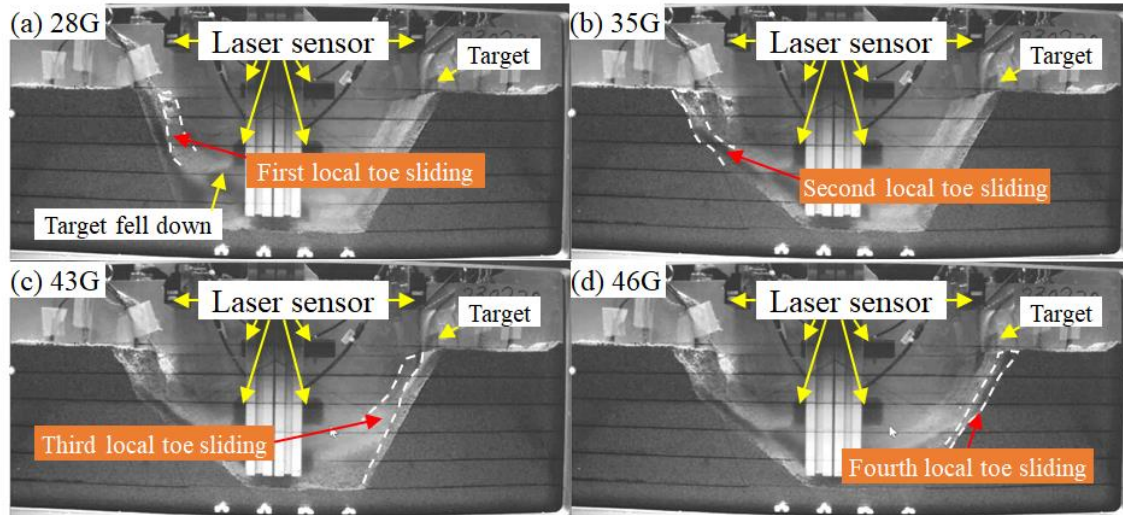


Fig. 5-28 Sliding process in Case 1 of the second group: (a) first local toe sliding on the left slope; (b) second local toe sliding on the left slope; (c) third local toe sliding on the right slope; (d) fourth local toe sliding on the right slope

Displacement

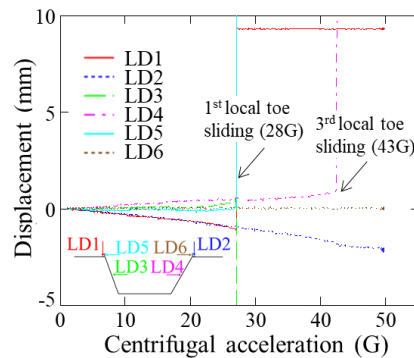


Fig. 5-29 Vertical and horizontal displacement of slope crests, and slope surfaces' horizontal displacement of Case 1 of the second group with increasing centrifugal acceleration

In **Fig. 5-29**, the abrupt changes in LD1 and LD5 at 28G were caused by the target falling on the left slope during the first local toe sliding, while the change in LD3 at that moment, which monitors the the left slope surface's horizontal displacement, was caused by sliding soil. Similarly, the increase in LD4 at 43G was induced by sliding soil during the third local toe sliding.

Earth pressure

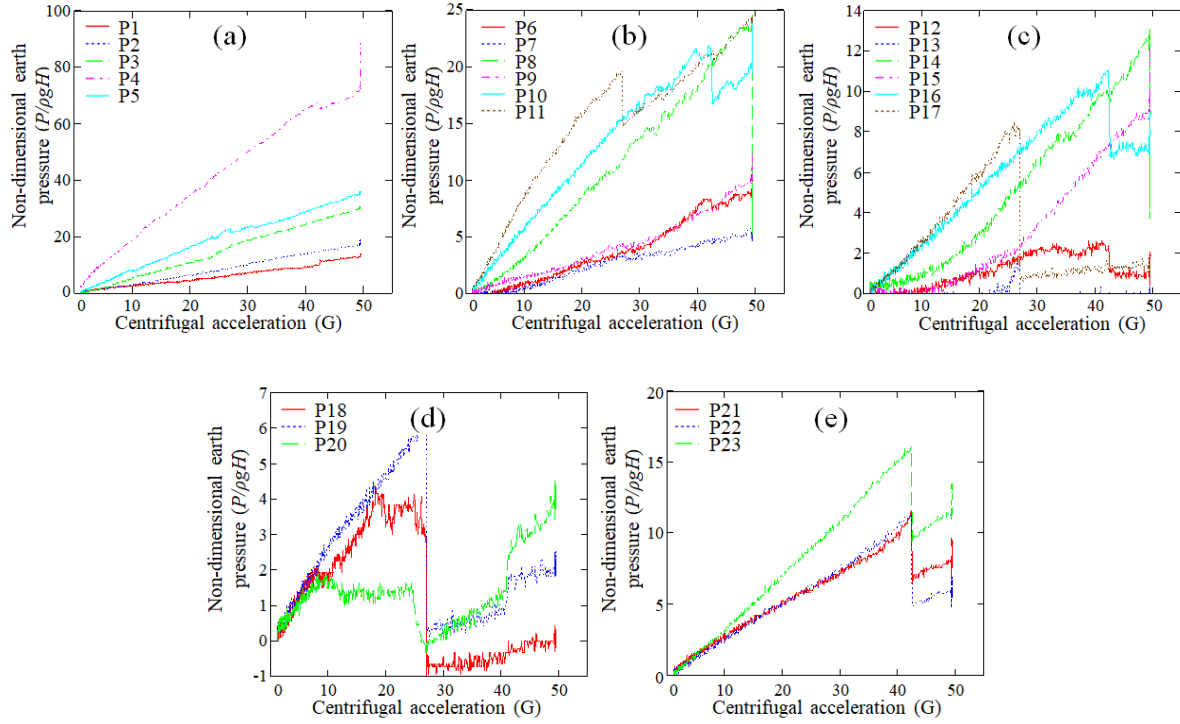


Fig. 5-30 Measured earth pressure with increasing centrifugal acceleration in Case 1 of the second group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope

Fig. 5-30 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 1 of the second group.

5.3.2.2 Case 2 ($\beta_L=70^\circ$, $\beta_R=60.58^\circ$, $E_{HL}=7.5$ cm, $E_{WL}=3$ cm, $E_{HR}=0$ cm, $E_{WR}=3$ cm)

Sliding process

In Case 2 of the second group, the left slope was benched by removing a parallelogram-shaped section from the slope crest, while the slope reduction method was applied on the right slope. Both sides had the same weight reduction, achieved through distinct geometric modifications compared to the original slope. The first bench sliding happened in the central area of the left slope's upper part at 20G (see **Fig. 5-31 (a)**). A toe sliding event followed on the left slope at 64G (see **Fig. 5-31 (b)**). At 68G, a local toe sliding took place near the back wall on the right slope (see **Fig. 5-31 (c)**), with a complete toe sliding surface forming on the right slope by 98G (see **Fig. 5-31 (d)**).

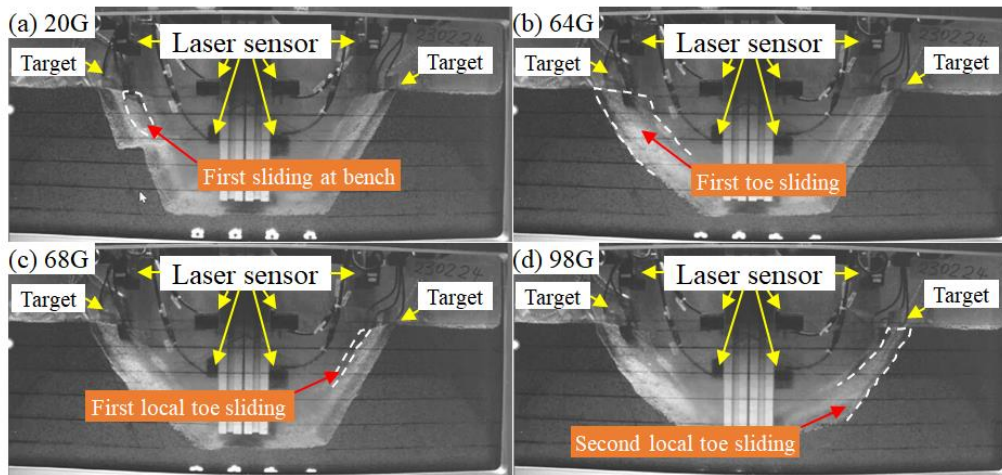


Fig. 5-31 Sliding process in Case 2 of the second group: (a) first sliding at bench on the left slope; (b) first toe sliding on the left slope; (c) first local toe sliding on the right slope; (d) second local toe sliding on the right slope

Displacement

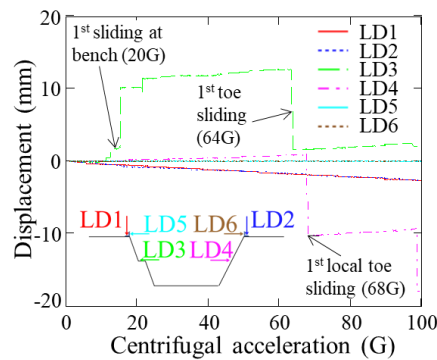


Fig. 5-32 Vertical and horizontal displacement of slope crests, and slope surfaces' horizontal displacement of Case 2 of the second group with increasing centrifugal acceleration

In **Fig. 5-32**, the increase in LD3 around 20G was caused by soil sliding during the first sliding event at the left slope's second bench. At 64G, the LD3 value dropped due to the first toe sliding. At 68G, the decrease in LD4 was induced by the first toe sliding on the right slope.

Earth pressure

Fig. 5-33 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 2 of the second group. As shown in **Fig. 5-33 (f)**, the values of P24, P26, and P28, which measure vertical earth pressure at the interface between the first bench and the main body of the slope, dropped at 64G when toe sliding occurred. In contrast, the other three gauges, which measure horizontal earth pressure, did not exhibit this response.

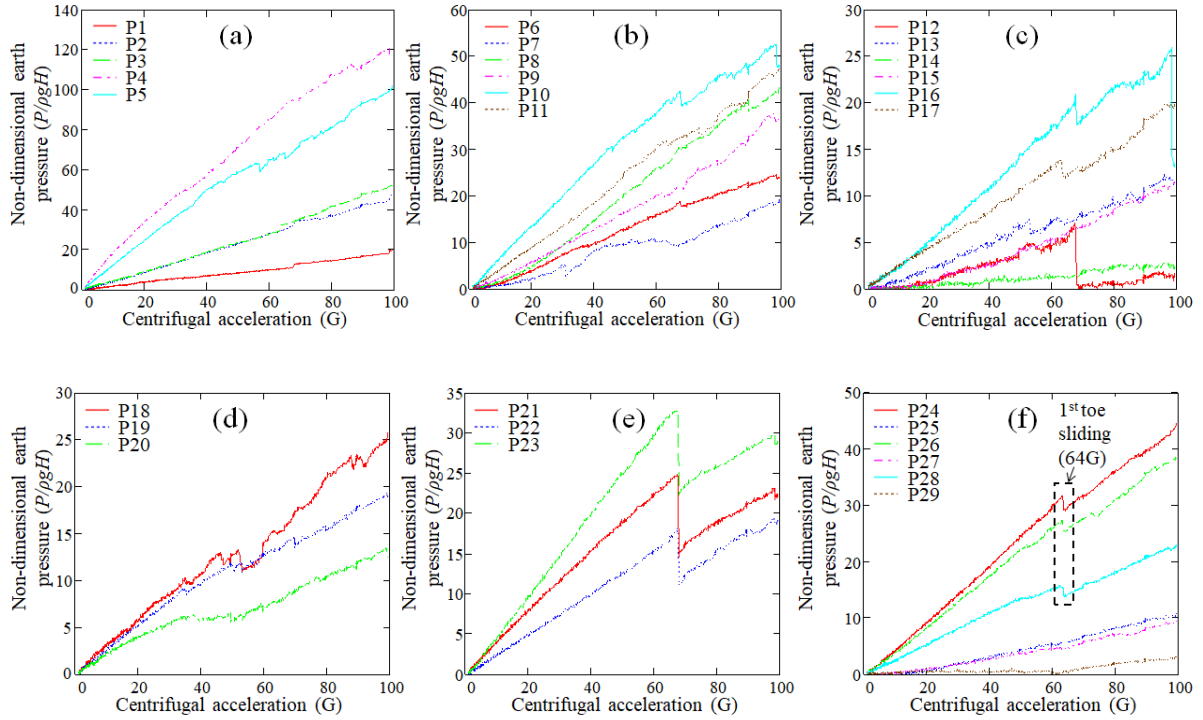


Fig. 5-33 Measured earth pressure with increasing centrifugal acceleration in Case 2 of the second group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope; (f) earth pressure between the slope and the first bench on the left hand side

5.3.2.3 Case 3 ($\beta_L=70^\circ$, $\beta_R=70^\circ$, $E_{HL}=7.5$ cm, $E_{WL}=3$ cm, $E_{HR}=3$ cm, $E_{WR}=7.5$ cm)

Sliding process

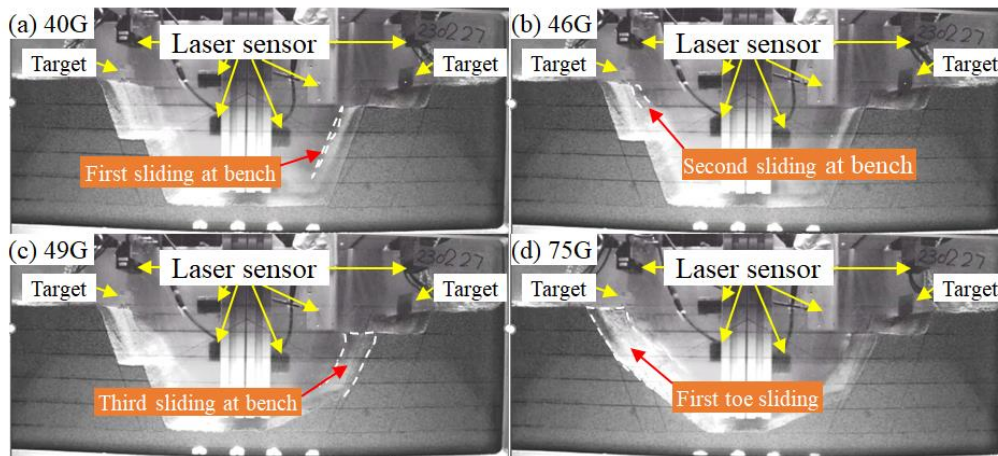


Fig. 5-34 Sliding process in Case 3 of the second group: (a) first sliding at bench on the right slope; (b) second sliding at bench on the left slope; (c) third sliding at bench on the right slope; (d) first toe sliding on the left slope

In Case 3 of the second group, both slopes were benched by removing two parallelogram-shaped sections, with the height of one matching the width of the other, and vice versa. A bench sliding happened at the right slope's first bench near the back wall at 40G (see **Fig. 5-34 (a)**). Another bench sliding then took place on the crest of the left slope near the back wall at 46G (see **Fig. 5-34 (b)**). At 49G, the entire surface of the first bench on the right slope slid (see **Fig. 5-34 (c)**). Finally, a toe sliding occurred on the left slope at 75G (see **Fig. 5-34 (d)**).

Displacement

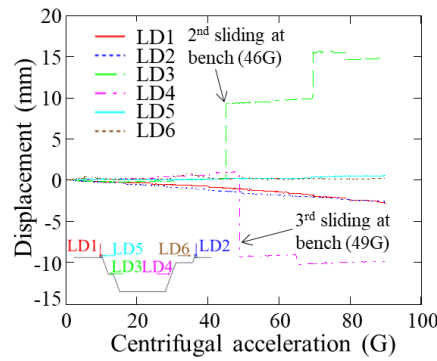
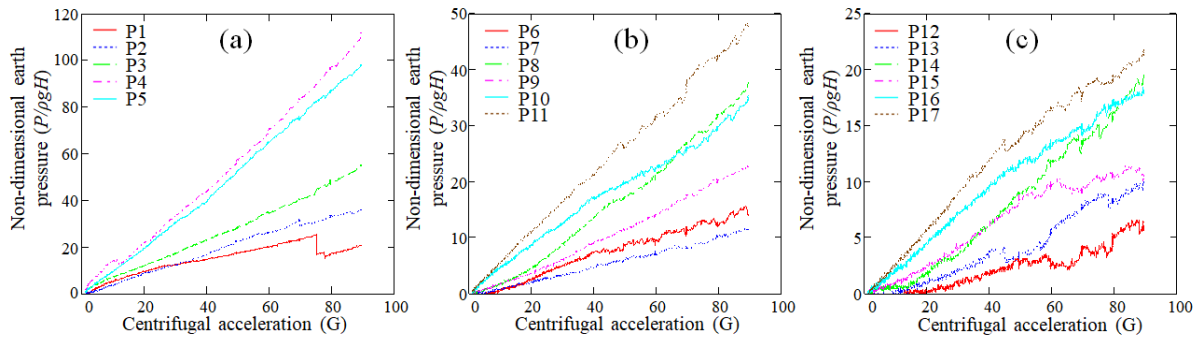


Fig. 5-35 Vertical and horizontal displacement of slope crests, and slope surfaces' horizontal displacement in Case 3 of the second group with increasing centrifugal acceleration

In **Fig. 5-35**, the increase in LD3 around 46G was caused by soil sliding during the second sliding event at the left slope's second bench. At 49G, the LD4 value dropped due to the third bench sliding on the right slope.

Earth pressure

Fig. 5-36 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 3 of the second group.



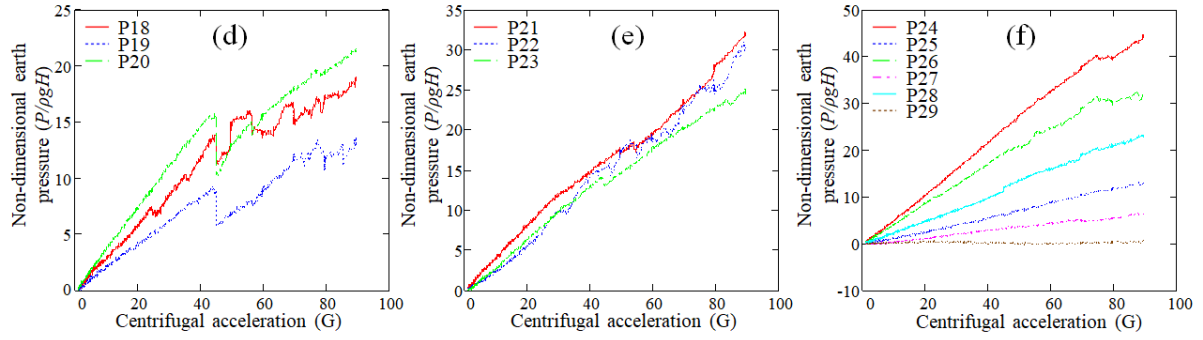


Fig. 5-36 Measured earth pressure with increasing centrifugal acceleration in Case 3 of the second group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope (f) earth pressure between the slope and the first bench on the left hand side

5.3.2.4 Case 4 ($\beta_L=70^\circ$, $\beta_R=60.58^\circ$, $E_{HL}=7.5$ cm, $E_{WL}=3$ cm, $E_{HR}=0$ cm, $E_{WR}=3$ cm)

Sliding process

Case 4 was a repeated test of Case 2. At 62G, a bench sliding occurred on the left slope's second bench near the back wall (see **Fig. 5-37 (a)**), and at 78G, another bench sliding appeared on the left slope's first bench near the glass (see **Fig. 5-37 (b)**). A local toe sliding then occurred on the right slope near the back wall at 79G (see **Fig. 5-37 (c)**). The complete toe sliding surface on the right slope did not form until 95G (see **Fig. 5-37 (d)**). This repeated case demonstrated greater strength than Case 2.

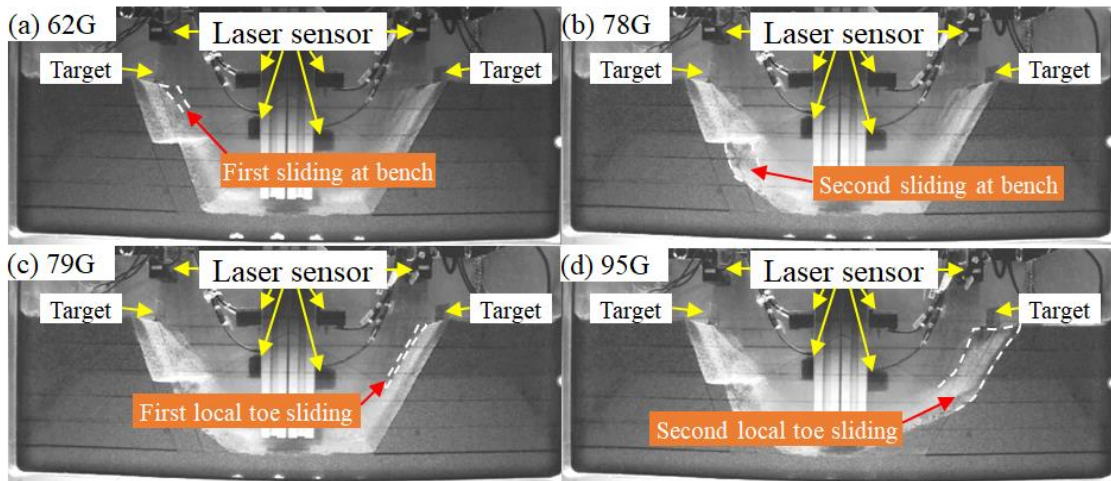


Fig. 5-37 Sliding process in Case 4 of the second group: (a) first sliding at bench on the left slope; (b) second sliding at bench on the left slope; (c) first local toe sliding on the right slope; (d) second local toe sliding on the right slope

Displacement

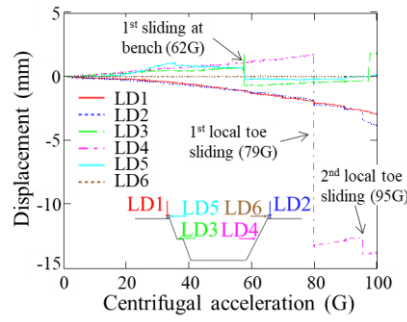


Fig. 5-38 Vertical and horizontal displacement of slope crests, and slope surfaces' horizontal displacement in Case 4 of the second group with increasing centrifugal acceleration

In **Fig. 5-38**, the decrease in LD3 and LD5 at 62G was caused by soil sliding during the first sliding event at the left slope's second bench. At 79G, the LD4 value dropped due to the first local toe sliding on the right slope, and the final local toe sliding at 95G caused an additional decrease in LD4.

Earth pressure

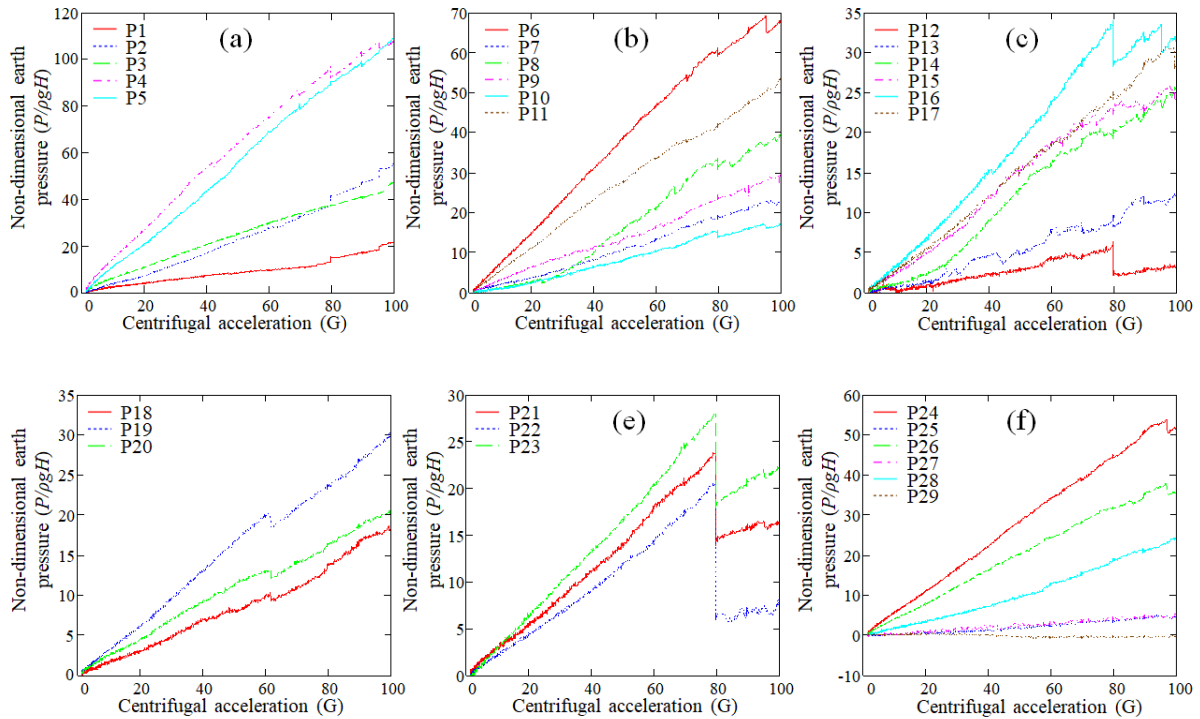


Fig. 5-39 Measured earth pressure with increasing centrifugal acceleration in Case 4 of the second group: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope (f) earth pressure between the left behind slope and the first bench

Fig. 5-39 shows the relationship between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 4 of the second group.

5.3.2.5 Case 5 ($\beta_L=70^\circ$, $\beta_R=70^\circ$, $E_{HL}=6$ cm, $E_{WL}=3.75$ cm, $E_{HR}=3.75$ cm, $E_{WR}=6$ cm)

Sliding process

In Case 5, both slopes were benched, with the dimensions of the excavated sections modified from Case 3 while maintaining the same area. The first bench sliding occurred at the left slope's second bench near the back wall at 39G (see **Fig. 5-40 (a)**). At 44G, a bench sliding took place at the right slope's first bench near the back wall (see **Fig. 5-40 (b)**). The third bench sliding appeared at the left slope's first bench at 45G (see **Fig. 5-40 (c)**). The sliding area expanded to cover the right slope's entire first bench at 54G (see **Fig. 5-40 (d)**), followed by a sliding of the entire left first bench at 57G (see **Fig. 5-40 (e)**). An accidental centrifuge stop occurred at 66G, and the test was subsequently restarted. During the second loading, toe sliding developed on the left slope at 90G (see **Fig. 5-40 (f)**).

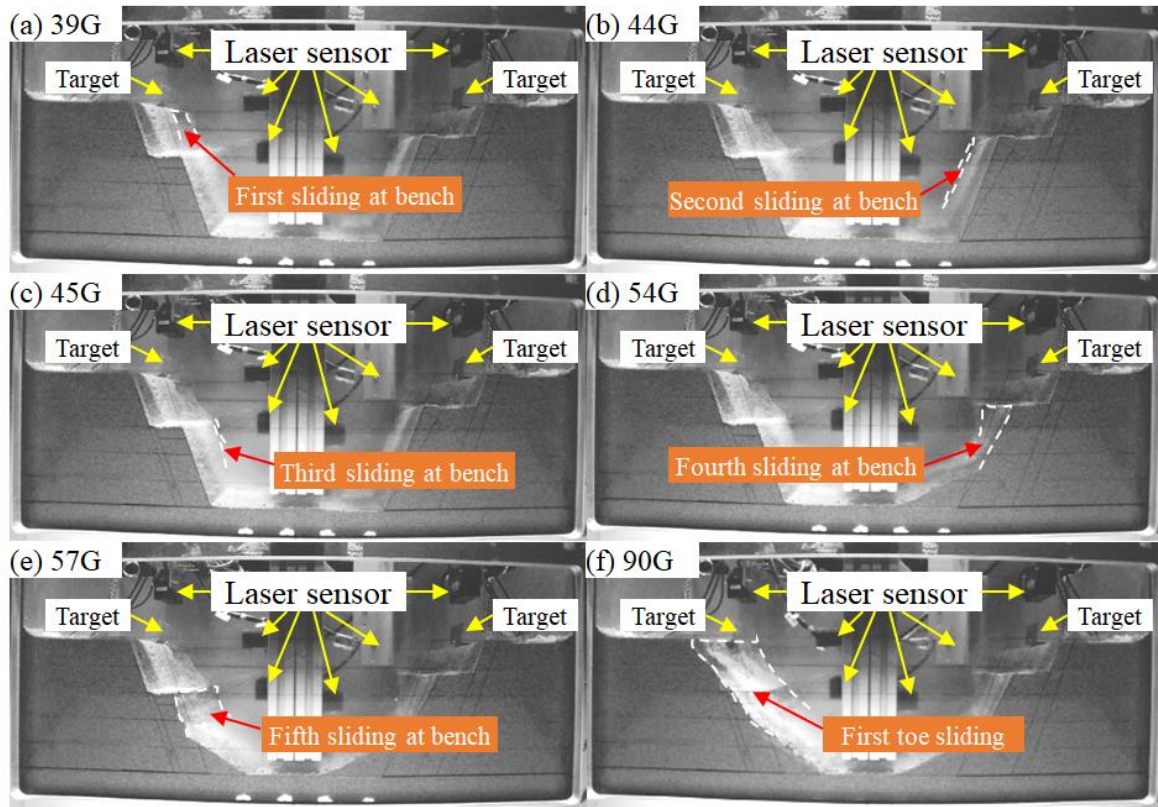


Fig. 5-40 Sliding process in Case 5 of the second group: (a) first sliding at bench on the left slope; (b) second sliding at bench on the right slope; (c) third sliding at bench on the left slope; (d) fourth sliding at bench on the right slope; (e) fifth sliding at bench on the left slope; (f) first toe sliding on the left slope

Displacement

In **Fig. 5-41 (a)**, the increase in LD3 at 39G was caused by soil sliding during the first sliding event at the left slope's second bench. At 44G, LD4 dropped due to the second time of bench sliding of the right slope, immediately followed by an increase in LD3 as sliding soil moved during the third bench sliding on the left slope. At 54G, the fourth bench sliding on the right slope caused a decrease in LD4, while additional sliding soil increased LD3. During the second loading, the first toe sliding event increased the value of LD3 at 90G (see **Fig. 5-41 (b)**).

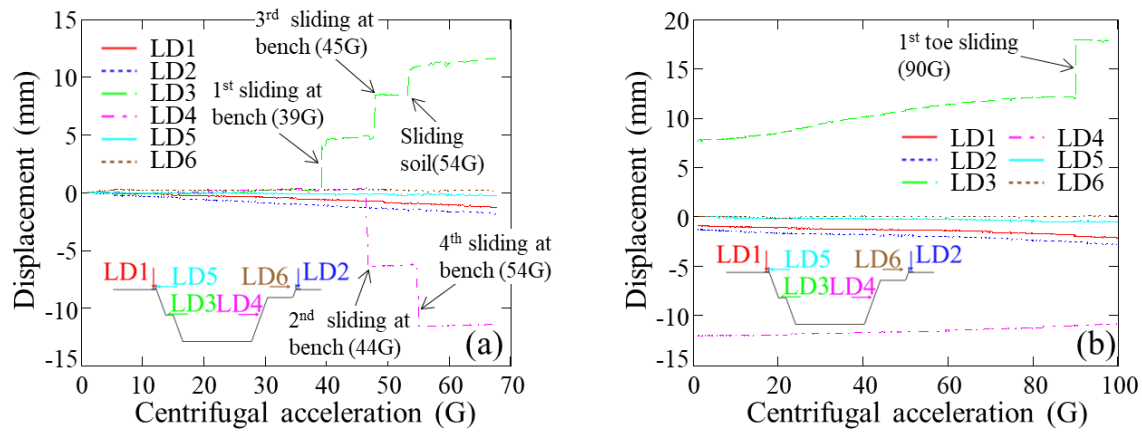
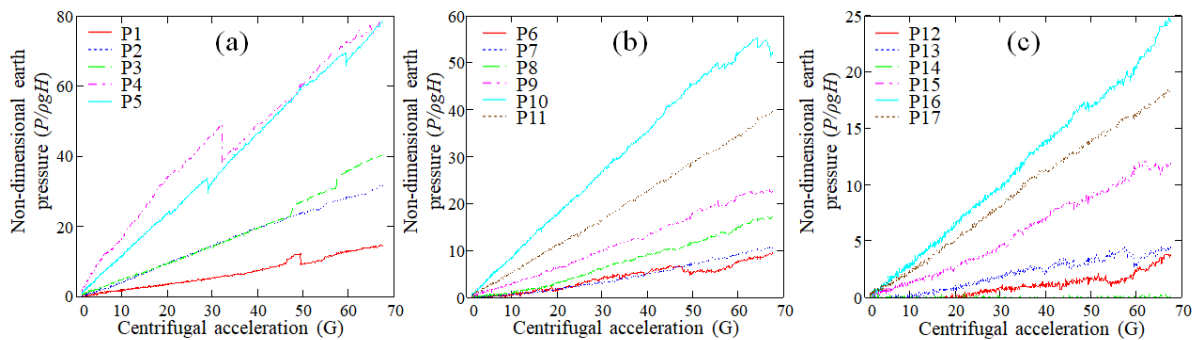


Fig. 5-41 Vertical and horizontal displacement of slope crests, and slope surfaces' horizontal displacement of Case 5 of the second group with increasing centrifugal acceleration: (a) during the first loading until 66G; (b) during the second loading until 100G

Earth pressure

Fig. 5-42 presents the correlation between non-dimensional earth pressures and centrifugal acceleration at three different heights in the model for Case 5 of the second group during the first loading.



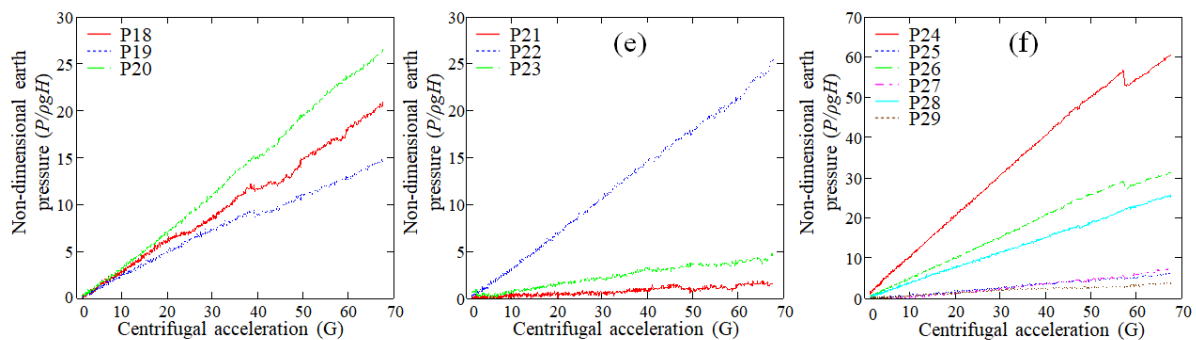


Fig. 5-42 Measured earth pressure with increasing centrifugal acceleration in Case 5 of the second group during the first loading to 66G: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope; (f) earth pressure between the left behind slope and the first bench

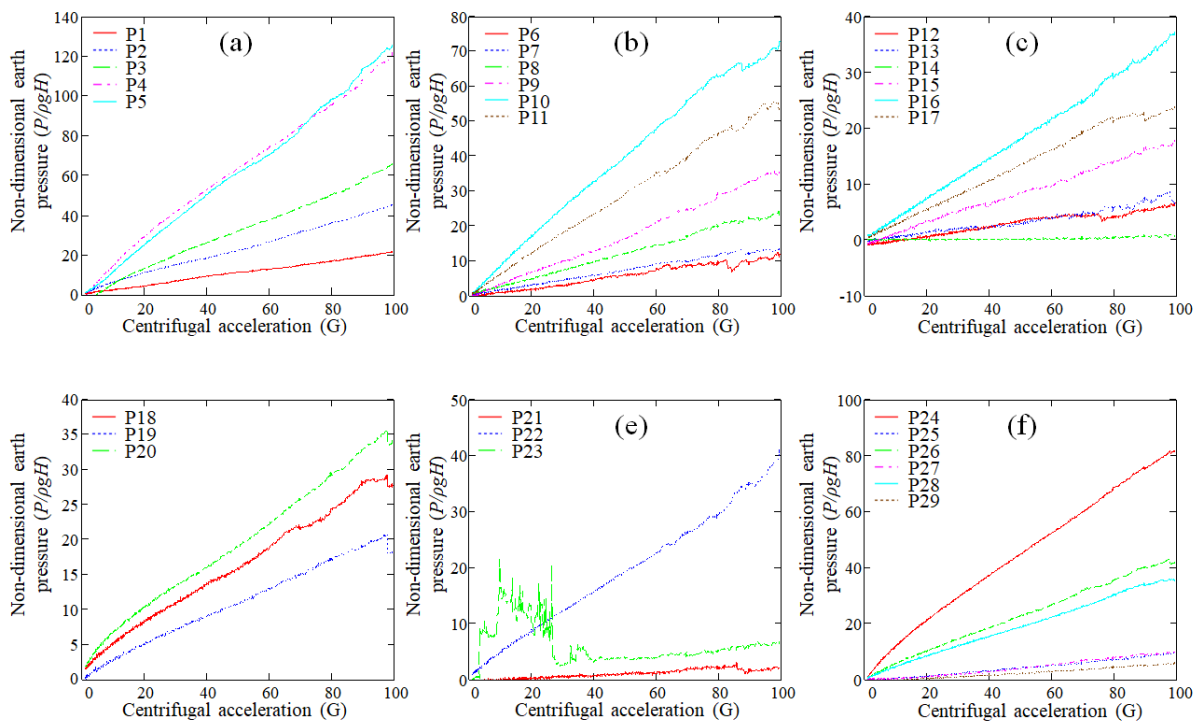


Fig. 5-43 Measured earth pressure with increasing centrifugal acceleration in Case 5 of the second group during the second loading to 100G: (a) bottom earth pressure; (b) earth pressure at one third height; (c) earth pressure at two thirds height; (d) earth pressure at the three-directional gauge in the left slope; (e) earth pressure at the three-directional gauge in the right slope (f) earth pressure between the left behind slope and the first bench

Fig. 5-43 shows the relationship between non-dimensional earth pressures and the centrifugal loading at three different model heights of Case 5 of the second group during the second loading.

The experimental results for all cases of the second group are listed in **Table 5-7**. After the testing, the water content in the model slopes was sampled from three distinct heights from the bottom of each model.

Table 5-7 Experimental results of the second group of tests

No.	Date	Final water content (%)			Sliding (G)			
		Bottom	Middle	Top	Left		Right	
		(0-2 cm)	(8-10 cm)	(16-18 cm)	First	Last	First	Last
1	230220	Δ	7.3	6.2	28	35	43	46
2	230224	Δ	9.1	6.3	20	64	68	98
3	230227	8.2	7.8	7	46	75	40	49
4	230301	Δ	8.7	6.6	62	95	79	95
5	230303	9.8	7.1	6.5	39	90	44	54

5.4 Discussion

5.4.1 Stability and failure modes of benched slopes in the test

5.4.1.1 First group of benched slopes with an additional bench

In the first group, the benched slope model was significantly safer compared to the uniform slope model with the same inclination, as observed in Case 1. In Cases 2 and 4, the slope with a higher first bench demonstrated greater stability. In Case 3, increasing the bench width also enhanced slope stability.

The failure pattern of the benched slope with a smaller first bench resembled a uniform slope's, with the sliding area connecting the slope crest and the toe. In contrast, for the benched slope having a bigger first bench, sliding was confined at the bench.

Since the main body of the slope behind the first bench was identical for both slopes in each case, the first bench can be considered a counterweight for the toe. A higher bench with greater mass provided additional resisting force to the slope section behind it. Additionally, the higher stability of the left second bench in all cases, despite having the same height as the first bench, may be attributed to variations in water content. After the experiment, samples from different heights of the model slope were measured (see **Table 5-6**). For example, in Case 4, the water contents at the top, middle, and bottom of the slope were 5.5%, 7.3%, and 9.2%, respectively. This distribution resulted from evaporation and gravitational forces during sample preparation, as well as centrifugal loading during testing. Such variation led to differences in shear strength across slope heights.

Another potential factor is that centrifugal loading was not uniform across the model height. The model's higher part, being closer to the centrifuge pivot, experienced less loading.

5.4.1.2 Second group of benched slopes with equivalent excavation

In the second group, uniform slopes were safer than benched slopes with the same excavation volume

from the original slope in Cases 2 and 4. This difference in stability is attributed to higher concentrated stress with a steep bench angle.

In Cases 3 and 5, both slopes were benched, resulting in a left first bench with a smaller bench height and width. Although a complete sliding surface appeared earlier on the right first bench, the crest of the right slope remained stable throughout the test. Therefore, distinct retrofit plans can be adopted based on different priorities. For the safety of personnel and equipment near the slope toe, the left side is preferable, as it requires a higher centrifugal acceleration to trigger sliding with a larger amount of sliding soil. However, for buildings' stability on the slope crest, the right bench configuration is more suitable.

5.4.2 Overburden pressure at the bottom

For each case, five earth pressure sensors (P1 to P5) were positioned along the longitudinal centerline at the model bottom to measure variations in overburden pressure under centrifugal loading. Sensor P1 was placed beneath the center of the model. Sensors P2 and P3 were positioned vertically below the slope toes, regardless of the first bench. Sensors P4 and P5 were located vertically below the slope crests.

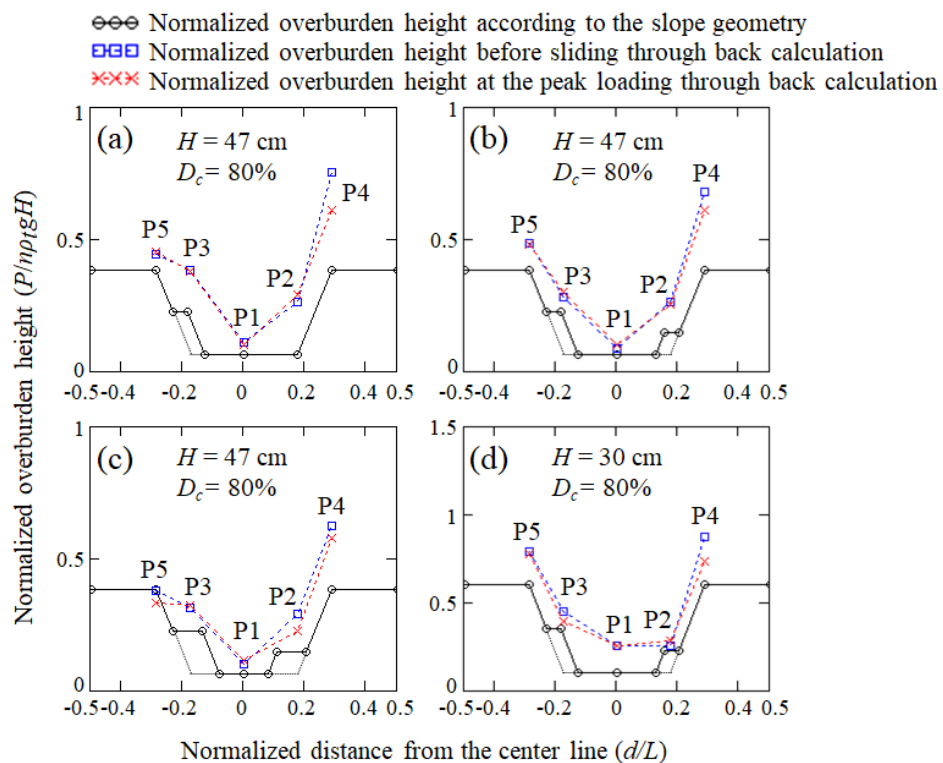


Fig. 5-44 Normalized overburden height at bottom of the model of the first group before sliding and at the peak loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4

For cases of the first group, **Fig. 5-44** compares the normalized overburden height obtained through

back calculation, before (5G prior to the first sliding) and after sliding (at peak loading), with the normalized actual overburden height. In the stable state, the back-calculated overburden height beneath the center of the model (P1) was similar to the actual overburden height. At other positions, the back-calculated values when the slope remained stable were higher than the actual values. The pressure below the left slope toe (P3), which has a larger first bench, was noticeably higher than that below the right toe (P2). The overburden height directly below the right slope crest (P4) decreased at peak loading due to sliding. In contrast, the overburden height below the left slope crest (P5) in Cases 1, 2, and 4 remained unchanged, as the second benches in these cases stayed stable. However, in Case 3, the overburden height below the left slope crest decreased due to a sliding event on the second bench.

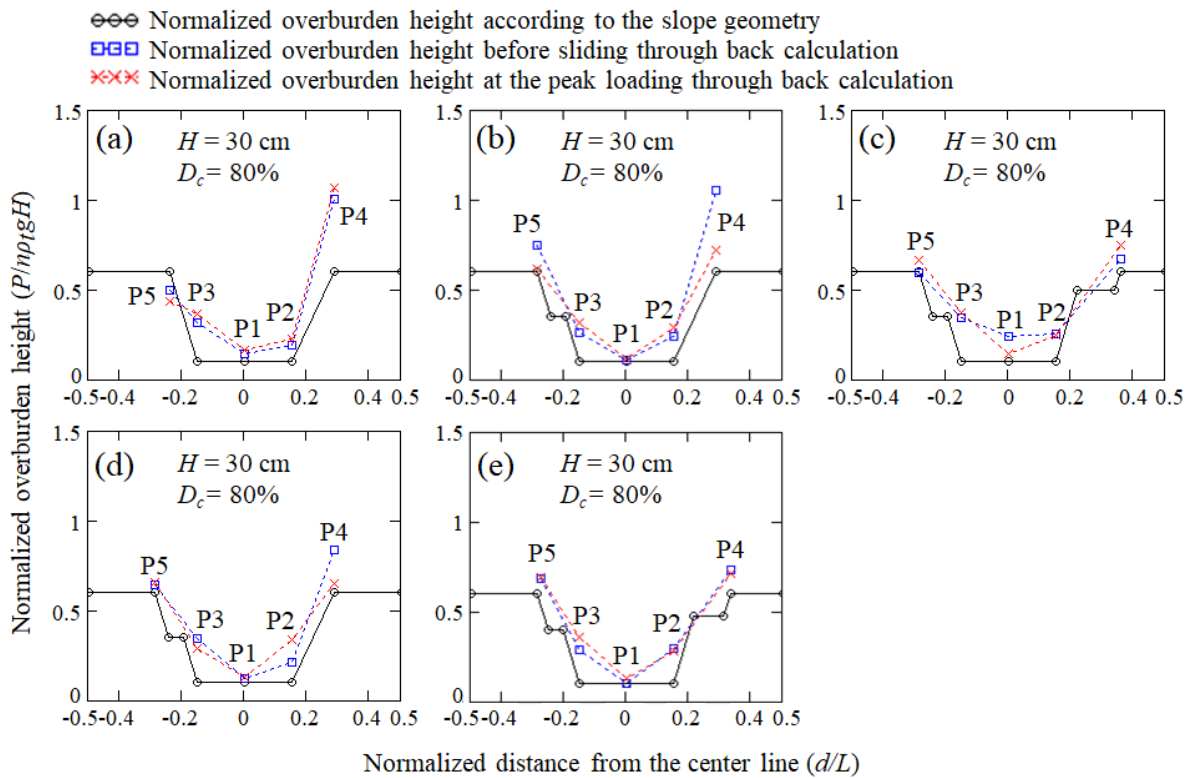


Fig. 5-45 Normalized overburden height at bottom of the model of the second group before sliding and at the peak loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5

For cases of the second group, **Fig. 5-45** compares the normalized overburden height obtained through back calculation, before and after sliding, with the normalized actual overburden height. In this group, P2 and P3 were positioned vertically below the toes of the first bench. In the stable state, the back-calculated overburden height beneath the center of the model (P1) was similar to the actual overburden height. At other positions, the back-calculated values when the slope remained stable were higher than the actual values.

In Case 1, the pressure below the left toe (P3) was greater than that below the right toe (P2) because the left slope was steeper than the right slope. In Cases 2 and 4, despite both slopes having the same

excavation weight, the steeper benched slope caused greater stress concentration at the toe, resulting in P3 being larger than P2. Similarly, in Cases 3 and 5, the benched slope with a narrower first bench width induced higher stress concentration at the toe, again making P3 larger than P2.

5.4.3 Comparison between experiments and theory

5.4.3.1 FS of the first group of benched slopes with an additional bench

To determine the contributions of the overburden part and the berm part. The FS is calculated for various configurations of the first bench with R equaling to zero in **Eq. (5-20)** and **Eq. (5-21)** first. The FS of each failure mode is plotted together with increasing centrifugal acceleration for each case in the first group as presented in **Fig. 5-46** to **Fig. 5-49**. The FS curve that first reaches a value of 1 is selected as the most likely failure pattern, with the upper 80% confidence interval represented. The experimental results are indicated by a vertical dashed line, marking the first sliding centrifugal acceleration.

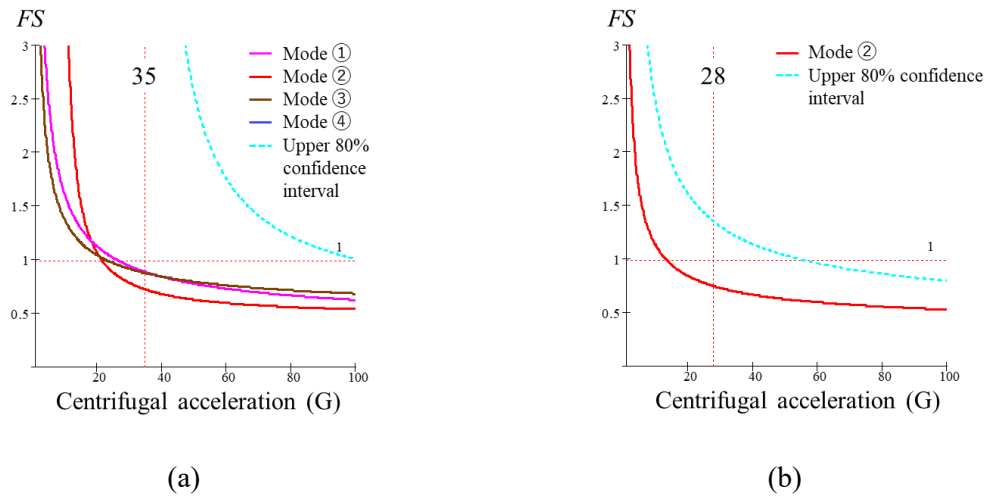


Fig. 5-46 Comparison between the prediction and experiments of Case 1 of the first group without considering the berm part: (a) left slope; (b) right slope

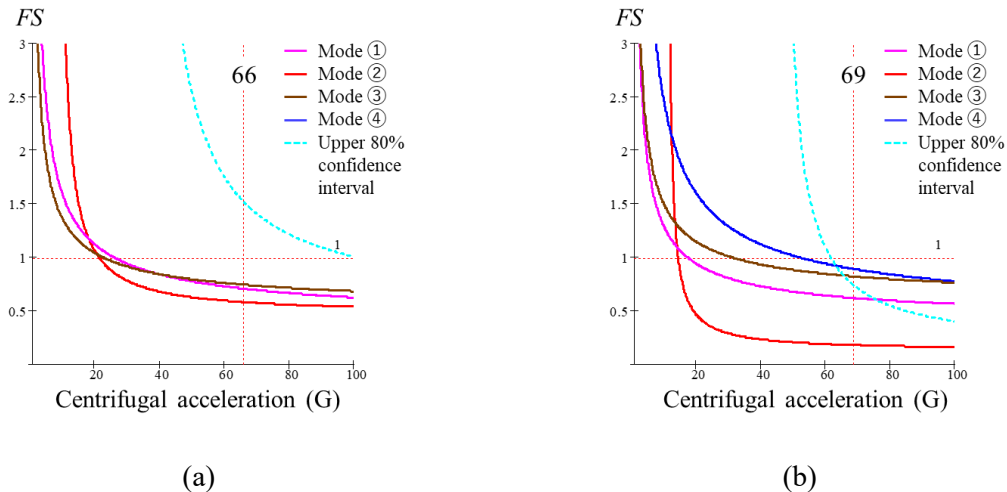


Fig. 5-47 Comparison between the prediction and experiments of Case 2 of the first group without considering the berm part: (a) left slope; (b) right slope

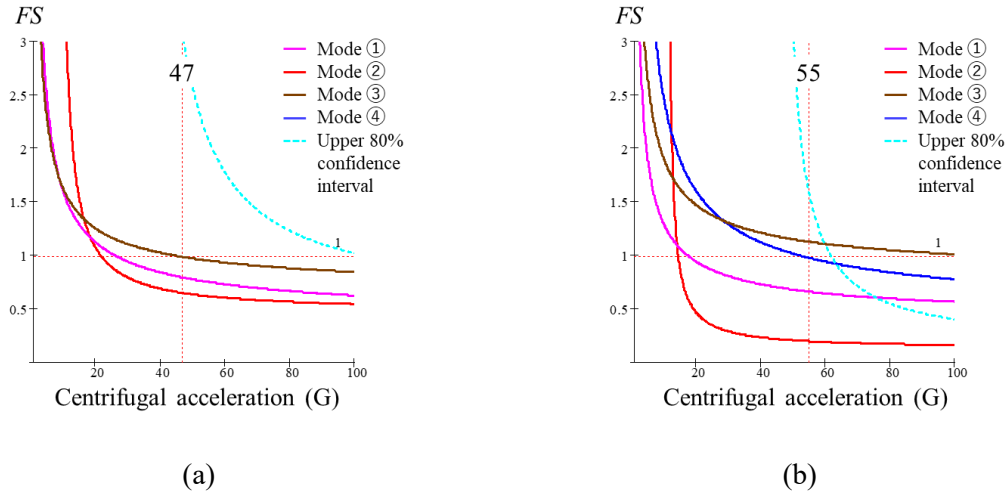


Fig. 5-48 Comparison between the prediction and experiments of Case 3 of the first group without considering the berm part: (a) left slope; (b) right slope

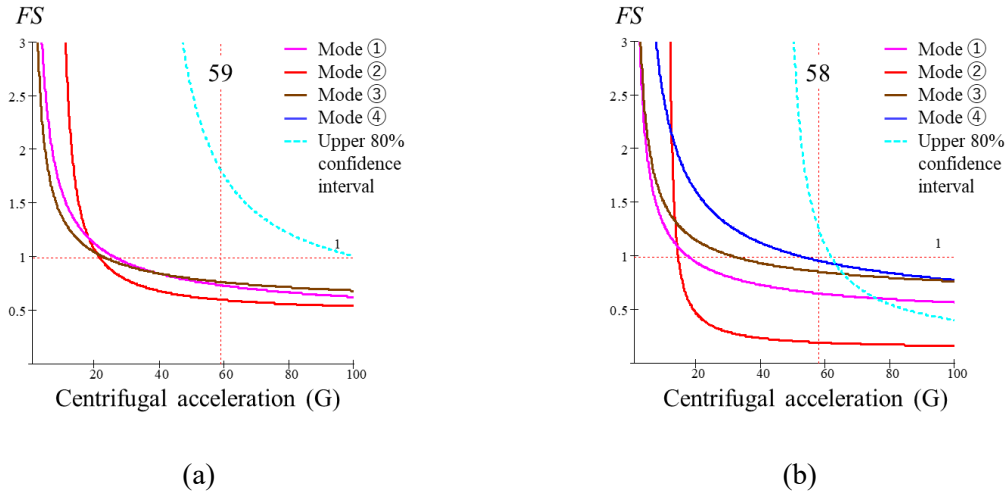
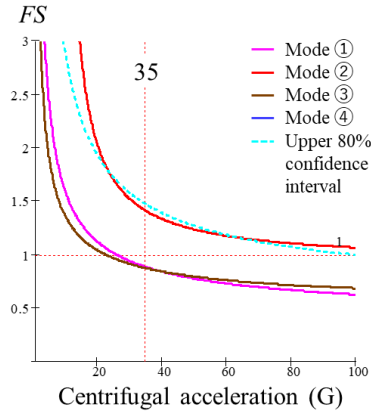
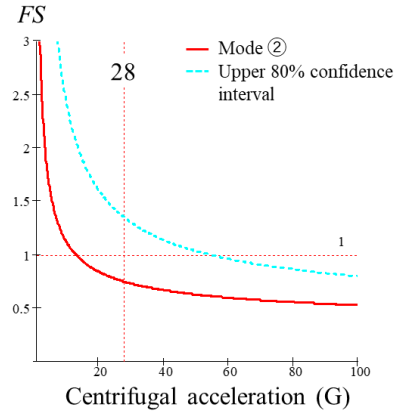


Fig. 5-49 Comparison between the prediction and experiments of Case 4 of the first group without considering the berm part: (a) left slope; (b) right slope

Except for the right slope of Case 2, as presented in **Fig. 5-47 (b)**, all experimental results fall within the predicted range. Theoretically, failure mode ② consistently appears as the most critical, reaching an FS of 1 first. However, this mode was rarely observed in the experiments, suggesting that the resisting force from R should be considered. The results, incorporating, R , are as follows.

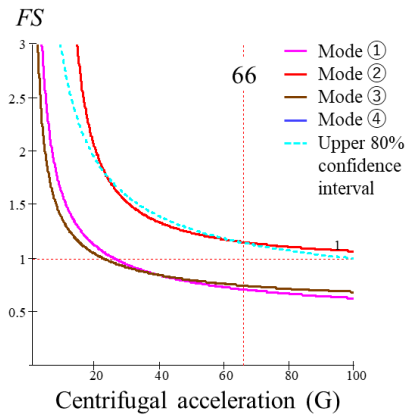


(a)

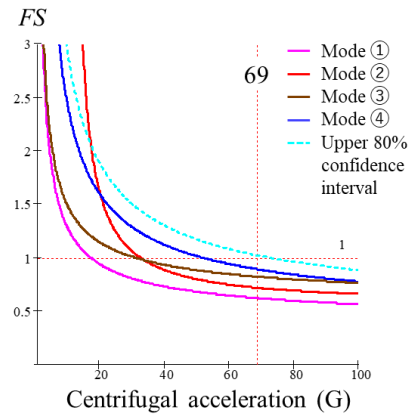


(b)

Fig. 5-50 Comparison between the prediction and experiments of Case 1 of the first group considering the berm part: (a) left slope; (b) right slope

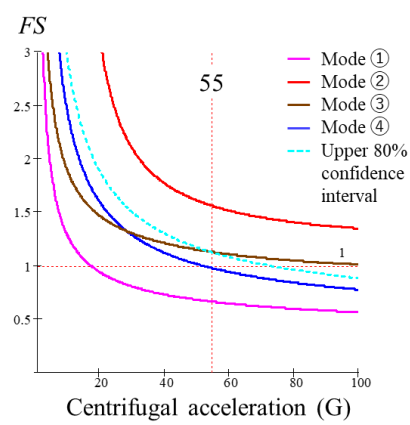
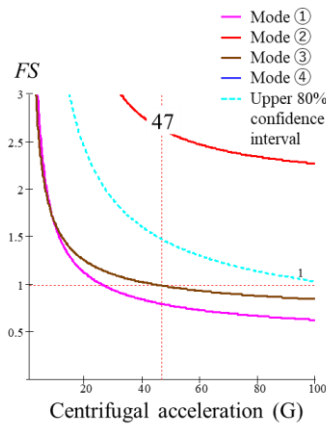


(a)



(b)

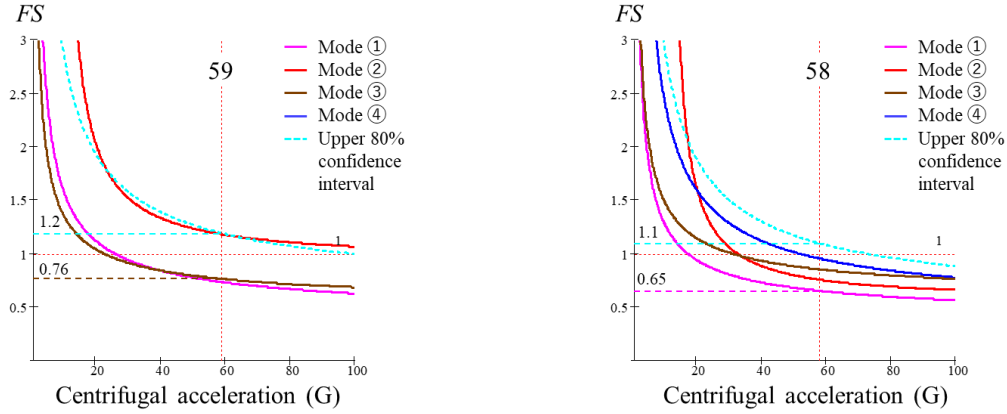
Fig. 5-51 Comparison between the prediction and experiments of Case 2 of the first group considering the berm part: (a) left slope; (b) right slope



(a)

(b)

Fig. 5-52 Comparison between the prediction and experiments of Case 3 of the first group considering the berm part: (a) left slope; (b) right slope

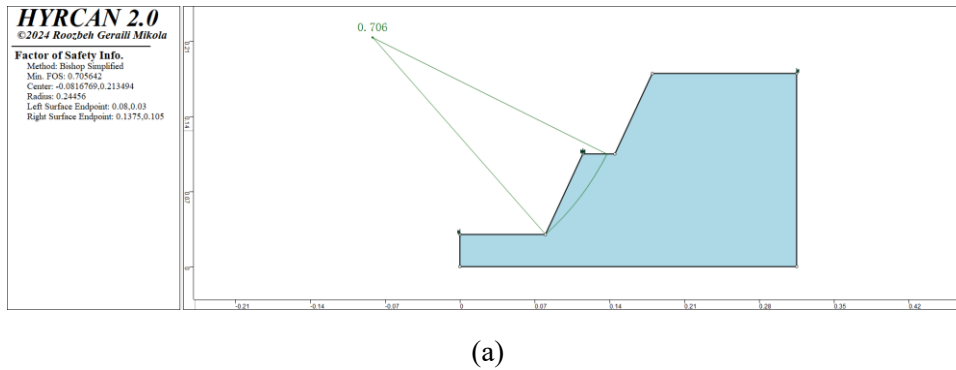


(a)

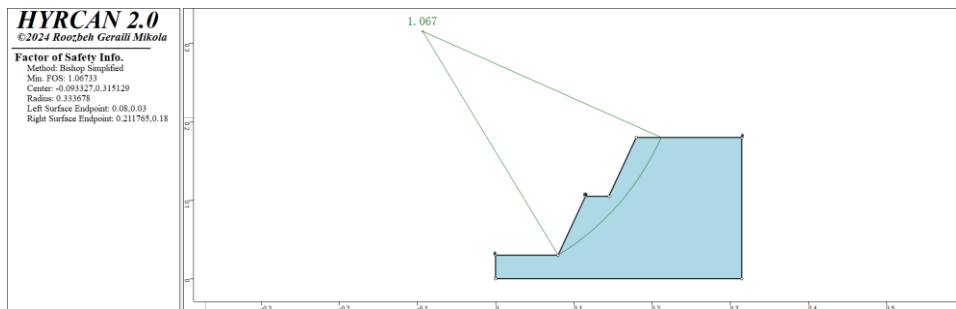
(b)

Fig. 5-53 Comparison between the prediction and experiments of Case 4 of the first group considering the berm part: (a) left slope; (b) right slope

In Fig. 5-50 to Fig. 5-53, all experimental results fall within the predicted range. Mode ① and mode ③ dominate the predicted failure mode, which align with most observed experimental outcomes. For the left slope in Cases 2 and 4, the upper bench did not fail as predicted, likely due to the decrease of moisture content along the slope height.



(a)



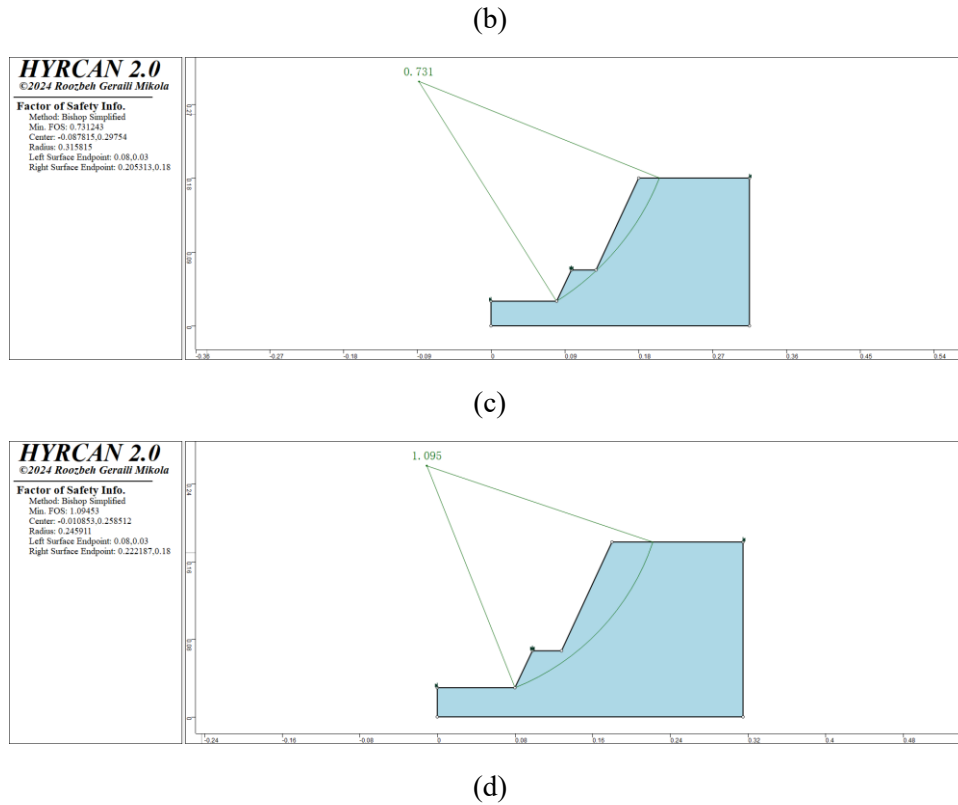
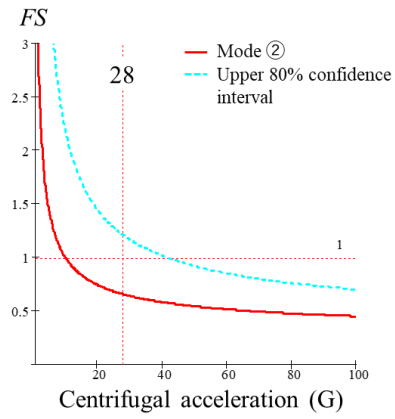


Fig. 5-54 *FS* calculated by Bishop Simplified method based on mean strength and upper 80% confidence level for Case 4: (a) left slope with mean strength; (b) left slope with upper 80% strength; (c) right slope with mean strength; (d) right slope with upper 80% strength

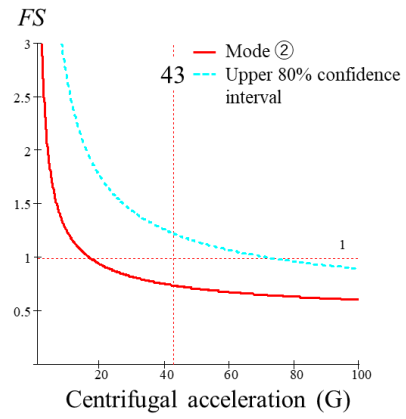
Using the HYRCAN software introduced in **Section 4.4.3**, predictions by 2D Culmann's method were compared with those from the Bishop Simplified method. Case 4 in **Fig. 5-53** was selected for comparison. For the left slope at failure, the *FS* predicted by Culmann's method was 0.76 and 1.2, using the mean strength and the upper 80% strength, respectively. Both values were higher than those obtained by the slice method, as shown in **Fig. 5-54 (a) and (b)**. Additionally, the most probable failure mode was mode identified was mode ③, which differed from the mode determined by the Bishop method using mean strength. For the left slope, Culmann's method yielded *FS* values of 0.65 and 1.1, with mode ① eing the most probable failure mode. In comparison, the Bishop method produced *FS* of 0.731 and 1.1, as shown in **Fig. 5-54 (c) and (d)**.

5.4.3.2 *FS* of the second group of benched slopes with equivalent excavation

As demonstrated in **Section 5.4.3.1**, the inclusion of *R* is necessary to accurately calculate *FS*. The *FS* for the second group of benched slopes with equivalent excavation is calculated accordingly. The results are presented as follows.

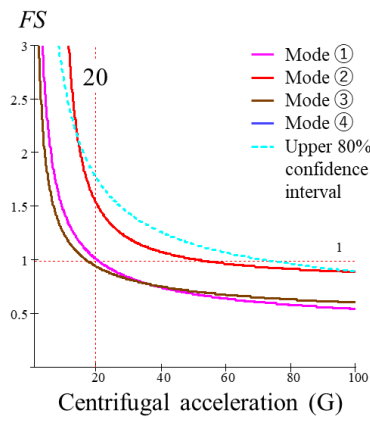


(a)

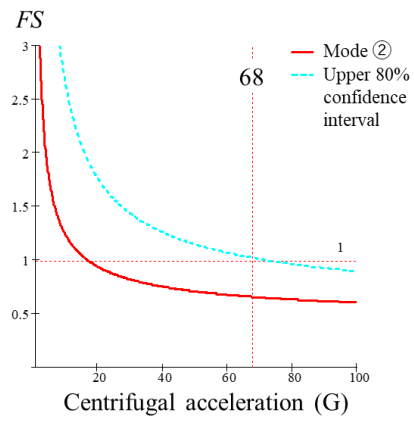


(b)

Fig. 5-55 Comparison between the prediction and experiments of Case 1 of the second group considering the berm part: (a) left slope; (b) right slope

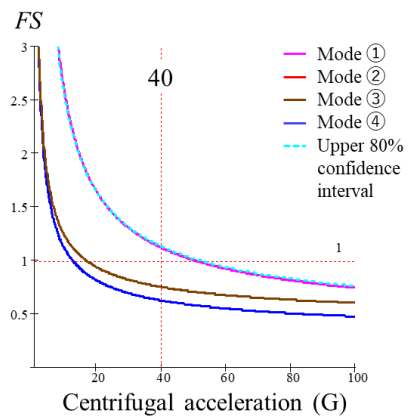
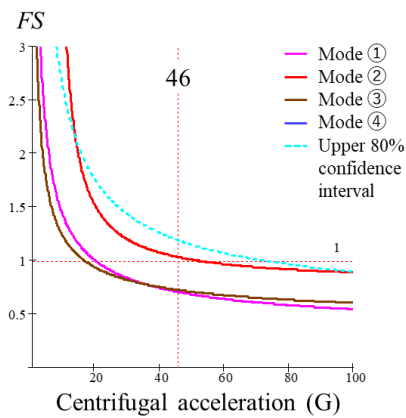


(a)



(b)

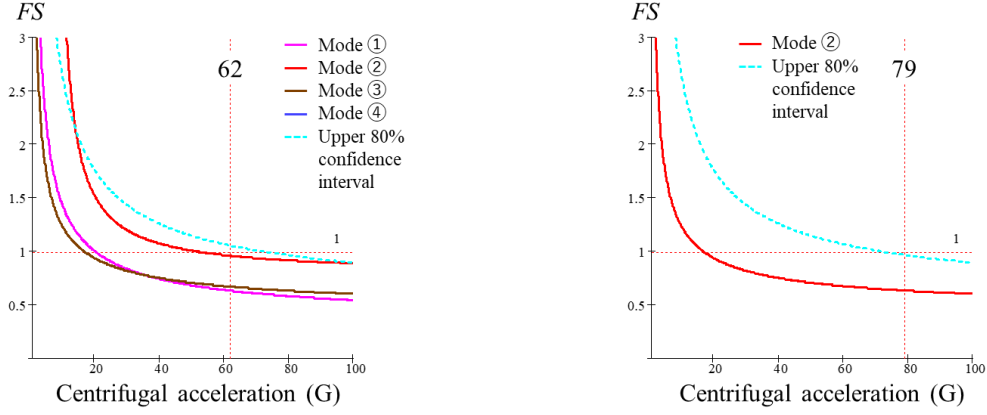
Fig. 5-56 Comparison between the prediction and experiments of Case 2 of the second group considering the berm part: (a) left slope; (b) right slope



(a)

(b)

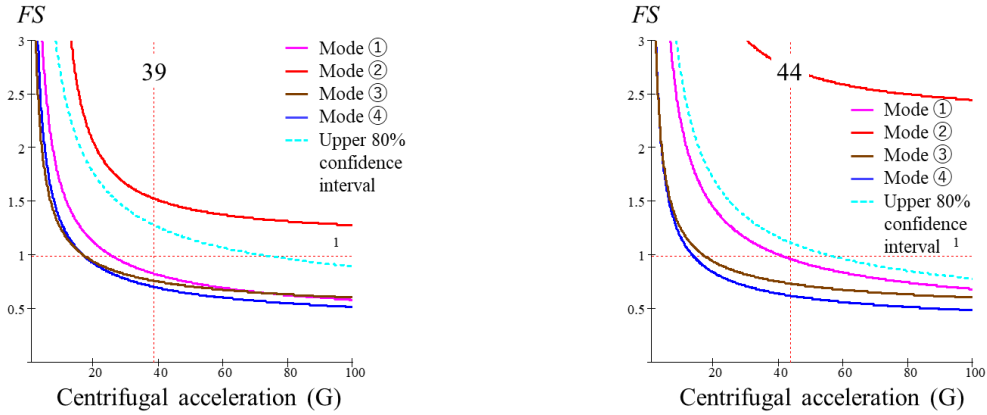
Fig. 5-57 Comparison between the prediction and experiments of Case 3 of the second group considering the berm part: (a) left slope; (b) right slope



(a)

(b)

Fig. 5-58 Comparison between the prediction and experiments of Case 4 of the second group considering the berm part: (a) left slope; (b) right slope



(a)

(b)

Fig. 5-59 Comparison between the prediction and experiments of Case 5 of the second group considering the berm part: (a) left slope; (b) right slope

From **Fig. 5-55** to **Fig. 5-59**, all experimental results fall within the prediction range. When the height of the excavated section is greater than its width in the left slope of Case 2, 3, 4 and 5, mode ③ is most likely to occur, as the *FS* of this mode decreases to 1 first, as shown in **Fig. 5-56 (a)**, **Fig. 5-57 (a)**, **Fig. 5-58 (a)** and **Fig. 5-59 (a)**. This suggests that these slopes' safety is comparable to a uniform slope with the same excavation. Conversely, when the height of the excavated part is less than its width, as in the right slopes of Cases 3 and 5, failure of the first bench becomes more likely. In these cases, the *FS* for

mode ④ declines faster than for mode ③, resulting in reduced slope stability.

5.4.4 Variation of the slope state with increasing centrifugal loading

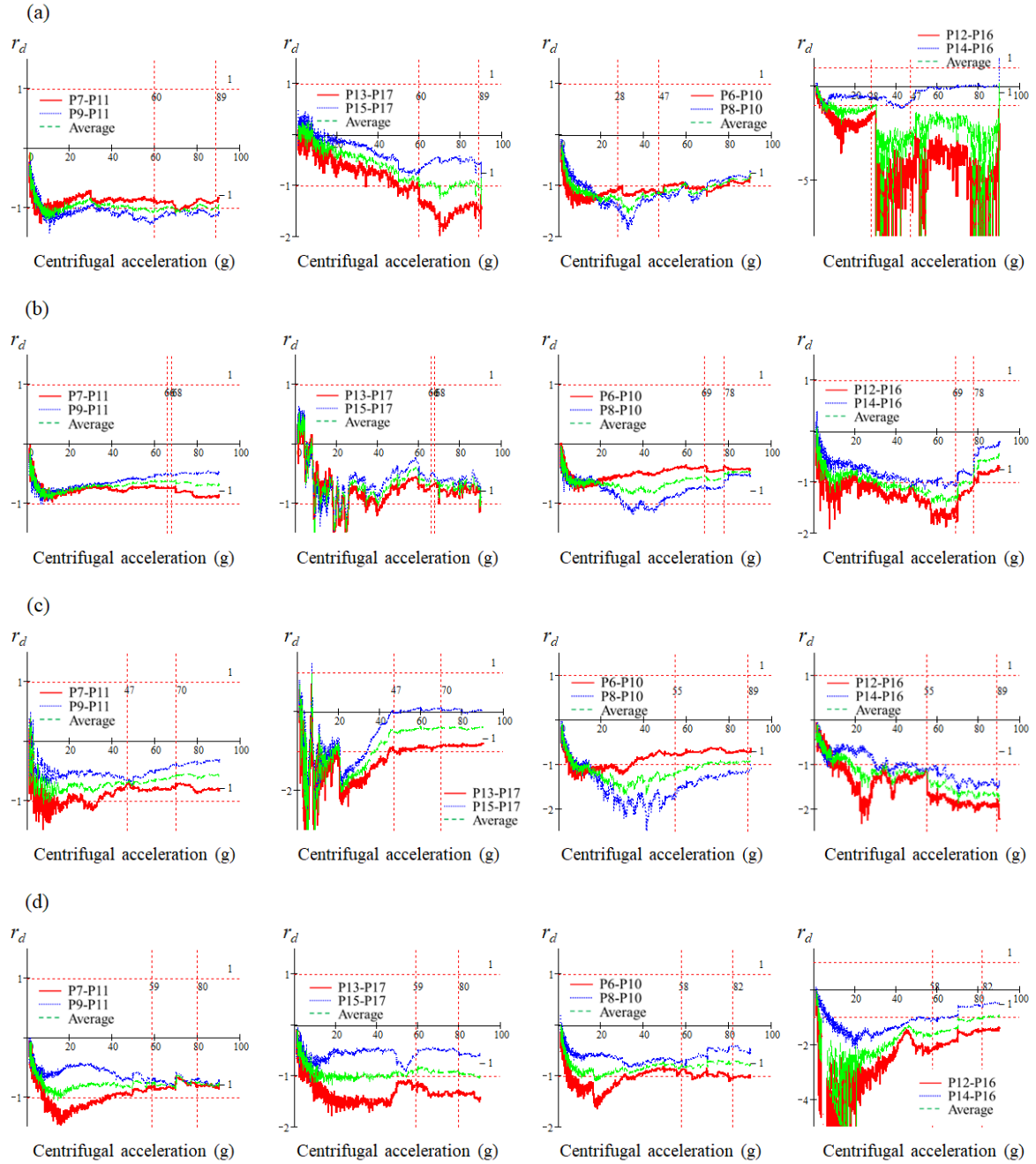


Fig. 5-60 Variation of reduction ratio with increasing centrifugal acceleration of the first group: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4

As mentioned in **section 4.1.3**, the reduction ratio, r_d , expressed by **Eq. (4-45)** can represent the slope state, ranging from the active state to a passive state. The model slopes deformed under the centrifugal loading during the test, leading to the varied states. For each slope, two r_d values can be calculated for

a given height, resulting in eight r_d . For example, in the right slope of Case 1, a horizontal earth pressure sensor (P10 on the center line of the model) and two vertical earth pressure sensors (P6 on the back wall, P8 on the right wall) are located 6 cm above the bottom of the soil chamber, hence, by combining P10 with P6 and P8 two r_d can be obtained (Tao & Pipatpongsa, 2023).

Fig. 5-60 shows variation of reduction ratios with increasing centrifugal acceleration for the first group. The average curve for the higher positions (P13/P15-P17; P12/P14-P16) decreases more than for the lower positions (P7/P9-P11; P6/P8-P10), because the slope section behind the bench is more stable than the area without bench. Additionally, the curves on the right side (P6/P8-P10; P12/P14-P16) decrease more than those on the left (P7/P9-P11; P13/P15-P17) at the same level, indicating that the slope with a lower bench is less stable than the slope with a higher bench. Furthermore, the average curve changes abruptly when toe sliding occurs, although this effect is less noticeable on the left side when sliding is confined only to the bench area.

Case 2 and Case 4 had the same cross-section but different model breadths. Case 2 was conducted in a wide soil chamber, while Case 4 was in a narrow soil chamber. The right slope in Case 2 developed a complete toe sliding surface earlier than in Case 4, at 78G and 82G, respectively, indicating that the broader slope is less stable than the narrower slope. Since both models finally achieved the same lateral resistance limited by active earth pressure when r_d approached to or exceeded -1; therefore, the lateral resistance per breadth of a narrower slope is higher than a broader slope, implying a higher factor of safety.

According to the measured displacement for Case 2 and Case 4 in **Fig. 5-20** and **Fig. 5-26**, The vertical displacements at the middle point in the slope crest for both models, which developed to 2.5~3 mm resulted in an estimated shear strain of $3 \text{ mm}/(0.5 \times 40 \text{ cm})=1.5\%$ for slope model in Case 2 and $3 \text{ mm}/(0.5 \times 20 \text{ cm})=3\%$ for slope model in Case 4 (Tao & Pipatpongsa, 2023). Since active earth pressure is induced by the strain release of slope, the average curve of r_d for Case 2 in **Fig. 5-60 (b)** decreases more slowly than its counterpart in **Fig. 5-60 (d)** for Case 4. Due to a larger shear strain, the narrower slope underwent the yielding (r_d approached to active limit) earlier than the broader slope.

Fig. 5-61 shows the variation of the reduction ratio with increasing centrifugal acceleration for the second group. In Case 1, as presented in **Fig. 5-61 (a)**, the curves of the right slope (P6/P8-P10; P12/P14-P16) decrease less than those on the left (P7/P9-P11; P13/P15-P17) at the same level, indicating that the model with a higher inclination is less stable than the model with a lower inclination. Additionally, the average curve shows an abrupt change when toe sliding occurs on the uniform slopes in Cases 1, 2, and 3. This phenomenon is less obvious for benched slopes, where most sliding is confined to the bench area. An extraordinary phenomenon is observed at one-third of the right slope's height of Case 4, where the average r_d increases to 1 (see **Fig. 5-61 (d)**). The reason is unclear.

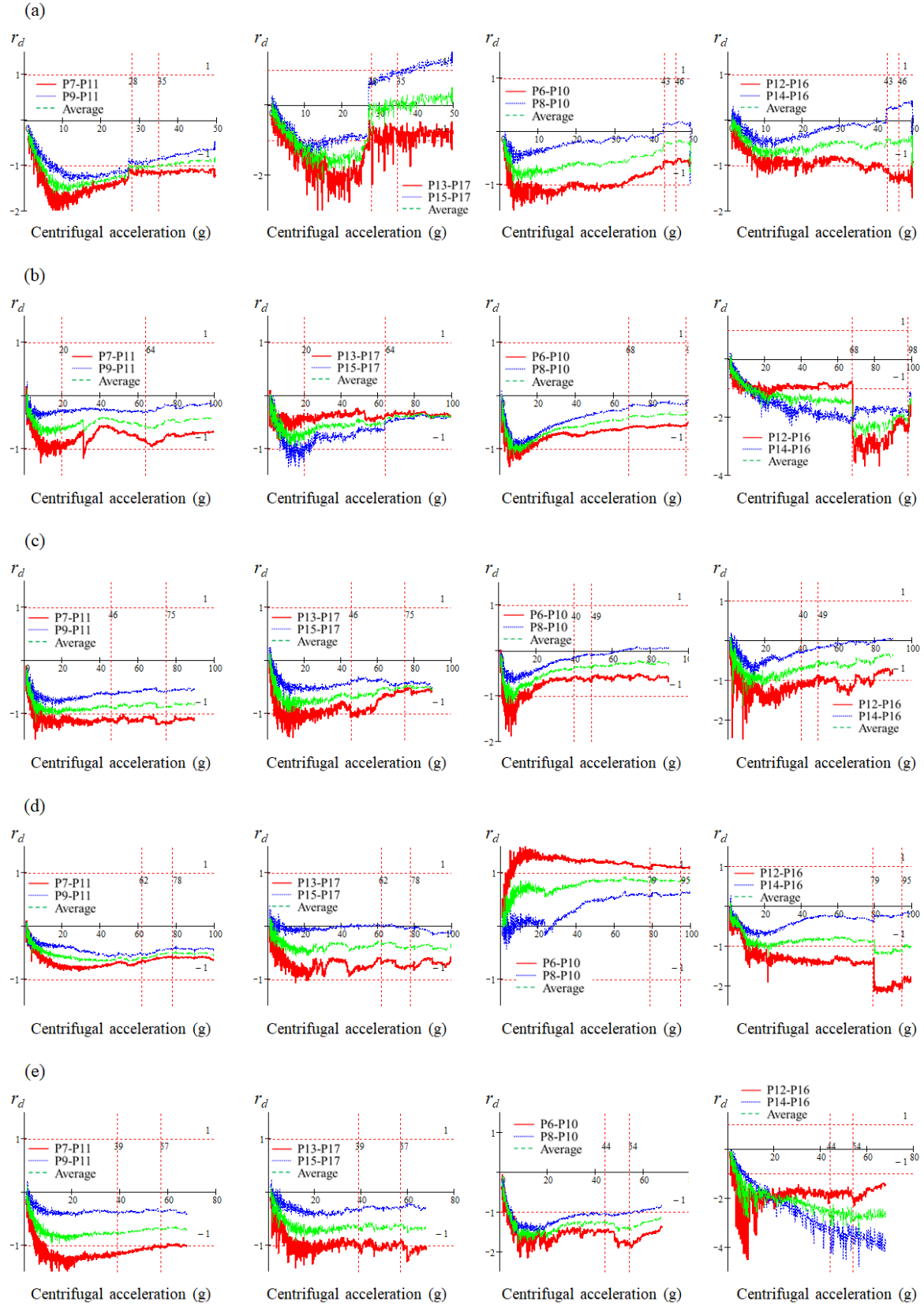


Fig. 5-61 Variation of reduction ratio with increasing centrifugal acceleration of the second group: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5

5.5 Conclusions

In this chapter, the benched slopes' stability was compared the uniform slopes' through centrifugal model tests. The slopes' failure modes altered by different bench geometries were observed and explained by comparing the variation in FS for each mode. Two types of bench design approaches were discussed from different perspectives. The major conclusions are listed as follows.

(1) Maintaining the slope inclination while adding a first bench at the slope toe can enhance the stability of the second bench. With an increase in the first bench's height and width, the failure mode shifts from toe sliding to bench sliding. As the failure mode can change, there is an optimal dimension for the first bench where the FS of multiple failure modes simultaneously reaches 1.

(2) In Culmann's method, only force equilibrium is considered, allowing for the application of equivalent excavation. However, experimental results indicated that while weight reduction increased slope stability, it did not guarantee the same FS due to variations in bench height and width. This discrepancy arises because moment equilibrium must also be considered. Although Culmann's method cannot precisely predict experimental results, it can still provide a conservative assessment of benched slope stability (Tao et al., 2023).

(3) For slopes with equivalent excavation from the original slope, benched slopes are less stable than uniform slopes. This is because the higher inclination of each bench, relative to the overall inclination, induces greater stress concentration. Additionally, toe sliding is more likely to occur when the excavated section's height is greater than its width; otherwise, bench sliding becomes the more dominant failure mode. Therefore, for a greater critical height of the entire slope, the former design is recommended. Conversely, the latter design is preferable for enhancing the slope crest's stability.

(4) Considering the first bench as a toe counterweight in failure mode ②, it is essential to account for both the overburden and berm components. Without these, this failure mode appears most likely to occur, contradicting the experimental results.

(5) The bench component can enhance the reduction factor of the behind slope, leading to a less active state and thereby enhancing the lateral resistance of the slope. However, as the entire slope remains in an active state, the lateral resistance is minimal. Therefore, a 2D stability method is sufficient for evaluation, considering the variability in shear strength influenced by the water content distribution.

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Chapter 6 Evaluation of dynamic stability of steep uniform slopes by centrifugal model tests

In this chapter, dynamic loading is integrated into Culmann's planar failure mechanism using a pseudo-static method to assess the stability of 2D finite uniform slopes. Boundary effects are also considered for 3D conditions, incorporating the reduction factor (r_d) (see **Section 6.1**).

Subsequently, a series of centrifugal model tests with uniform slope models are conducted to verify the proposed theories. The experimental scheme, introduced in **Section 6.2**, includes five cases involving eight uniform model slopes. The material used, Hiroshima sand, was tested in **Chapter 3**.

In **Section 6.3**, the horizontal seismic coefficients at the onset of shallow sliding and the observed failure modes of model slopes are recorded. Additionally, data from installed sensors is presented to illustrate variations in displacement and earth pressure of the slopes during shaking.

The experimental results are then compared with the pseudo-static Culmann's method under 2D conditions in **Section 6.4**. The slope states, evaluated by r_d with increasing shaking intensity, are also discussed, followed by a 3D theoretical analysis. Additionally, the seismic amplification phenomenon, resulting in distinct stress-strain relationships, is illustrated for a selected case.

Lastly, the findings presented in this part are listed in **Section 6.5**.

6.1 Theoretical consideration

6.1.1 Pseudo-static Culmann's method under 2D conditions

For slopes under seismic conditions, the inertia force induced by seismic wave must be considered. Based on the 2D Culmann's theory for evaluating a sliding wedge's stability, the safety of a uniform slope can be studied by incorporating the seismic loading using the pseudo-static theory. For simplicity, only the seismic wave in the horizontal direction is considered here.

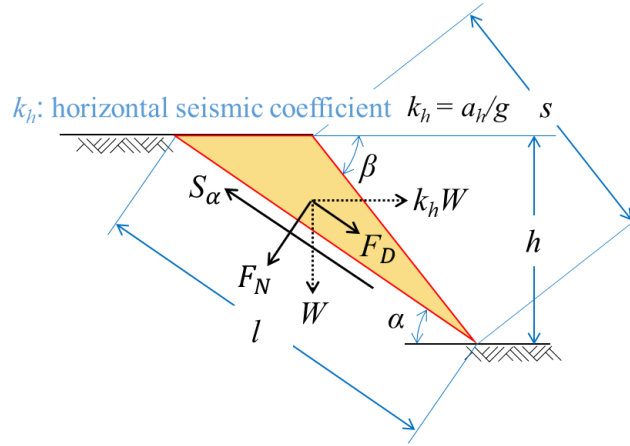


Fig. 6-1 Schematics of Culmann's method with pseudo-static force

Assuming a failure plane passing the slope toe, the force diagram of a possible wedge-shaped sliding block is illustrated in **Fig. 6-1** under 2D conditions. Along the slope direction, the driving force F_D consists of wedge's weight component, W , and inertia force's component, $k_h W$ (k_h : horizontal pseudo-static seismic coefficient). The resistance, S_α , is produced by frictional and cohesive forces along the entire failure surface according to the Mohr-Coulomb failure criterion. Angles of the slope and the sliding plane are β and α , respectively. The expressions for the slope surface's length, s , and the failure plane's length, l , are provided below.

$$s = h \cdot \csc \beta \quad (6-1)$$

$$l = h \cdot \csc \alpha \quad (6-2)$$

$$W = \frac{1}{2} n \rho_t g h^2 \cdot (\cot \alpha - \cot \beta) \quad (6-3)$$

$$F_N = W(\cos \alpha - k_h \sin \alpha) \quad (6-4)$$

$$F_D = W(\sin \alpha + k_h \cos \alpha) \quad (6-5)$$

$$S_\alpha = cl + F_N \tan \phi \quad (6-6)$$

The FS along the sliding plane can be determined by **Eq. (6-7)**.

$$FS = \frac{S_a}{F_D} = \frac{2c}{\rho_i g h (\sin \alpha)^2 (\cot \alpha - \cot \beta) (1 + k_h \cot \alpha)} + \frac{(1 - k_h \tan \alpha) \tan \phi}{(1 + k_h \cot \alpha) \tan \alpha} \quad (6-7)$$

To determine the failure plane's angle, take the partial derivative of FS with α and set it to 0 to achieve minimization. After calculation, the resulting α is presented as a function of β , ϕ and k_h in **Eq. (6-9)**.

$$\frac{\partial FS}{\partial \alpha} = 0 \quad (6-8)$$

$$\alpha = \frac{\beta + \phi - \tan^{-1} k_h}{2} \quad (6-9)$$

When $FS = 1$, **Eq. (6-7)** and **Eq. (6-9)** yield the equation for n_f as shown in **Eq. (6-10)**.

$$n_f = 4c \cos(\tan^{-1} k_h) \frac{\sin\left(\frac{\beta + \phi + \tan^{-1} k_h}{2}\right)}{\rho_i g h \sin\left(\frac{\beta + \phi - \tan^{-1} k_h}{2}\right) \left(\cot\left(\frac{\beta + \phi - \tan^{-1} k_h}{2}\right) - \cot(\beta)\right) \left(\frac{\cos(\phi) - \cos(\beta + \tan^{-1} k_h)}{\cos(\phi)}\right)} \quad (6-10)$$

6.1.2 Pseudo-static Culmann's method under 3D conditions

Following the same derivation process as in **Section 4.1**, the effect of lateral resistance, related to the slope state, can be incorporated into the FS for slopes under 3D conditions. The resulting expression is provided in **Eq. (6-11)**.

$$FS = \frac{2c}{\rho_i g h (\sin \alpha)^2 (\cot \alpha - \cot \beta) (1 + k_h \cot \alpha)} + \frac{(1 - k_h \tan \alpha) \tan \phi}{(1 + k_h \cot \alpha) \tan \alpha} + \frac{h\sqrt{K}}{B(\sin \alpha + k_h \cos \alpha)} \left[\frac{2}{3} \sqrt{K} \left(1 - \frac{\cot \beta}{\cot \alpha} \right) + \frac{4r_d c}{\rho_i g h} \right] \tan \phi_i + \frac{2c_i}{\rho_i g B(\sin \alpha + k_h \cos \alpha)} \quad (6-11)$$

6.2 Experimental scheme

6.2.1 Test program

Centrifugal model tests were carried out to investigate the seismic stability of uniform slopes. The centrifuge facility utilized for these tests is located at the Geotechnical Centrifuge Center, Disaster Prevention Research Institute (DPRI), Kyoto University, featuring an effective radius of 2.5 m, maximum payload of 24 G×ton and can achieve a maximum centrifugal acceleration of 200G (Tao & Pipatpongsa, 2022). The attached shaking table to the centrifuge has a maximum capacity of 142.5 kg at 50G for dynamic excitation.

Five dynamic centrifugal models were constructed, comprising eight uniform model slopes and two model slopes with toe counterweights (left slopes in Case 3 and Case 4). For each case, two model

slopes were tested simultaneously with symmetric seismic excitation to enhance efficiency and allow observation of the effects of different slope angles. **Fig. 6-2** shows the constructed model slopes prior to loading, with all model slopes 9 m in a total height and 7.5 m in a slope height on the prototype scale at 50G. All cases were constructed using damp Hiroshima sand with $D_c=90\%$ and $w=8\%$. In particular, dry udon noodles were inserted into the model in Case 4 and Case 5 to visualize the sliding surface. Only the narrow soil chamber was used in the second group (see **Fig. 4-8**).

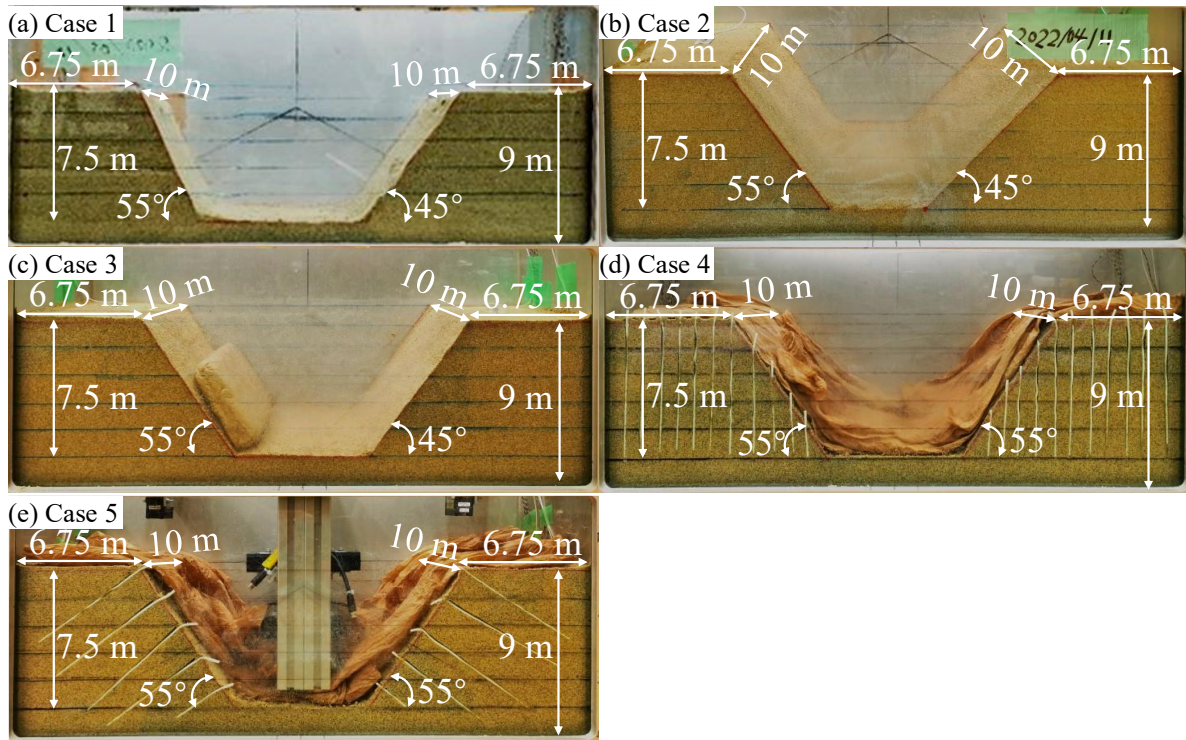


Fig. 6-2 Constructed model slopes before loading (prototype scale at 50G)

Table 6-1 Model configurations for uniform slopes subject to ground motions

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)		Initial water content, w (%)	Loading (G)	Shaking magnification
				Left	Right			
1	211022	Narrow	90	55	45	8	50	0.3→0.5→0.8→1.0→1.2→1.5→1.7→2.0→2.2→2.5→2.7
2	220412			55	45			1.3→1.4→1.5→1.9→2.0→2.2→2.5→2.7
3	220426			55	45			1.9→2.5→2.7→2.7→3.0
4	221007			55	55			1.9→2.5
5	221017			55	55			1.9→2.5

Table 6-1 presents the detailed model configurations for all five cases, including the case number, experimental date, type of soil chamber used, degree of compaction (D_c), slope angle (β), initial water

content (w), centrifugal loading, and the sequence of amplitude magnifications (to be explained later) for each test. A centrifugal loading of 50G was applied in all tests. **Fig. 6-3** provides a diagram illustrating a model slope under centrifugal loading.

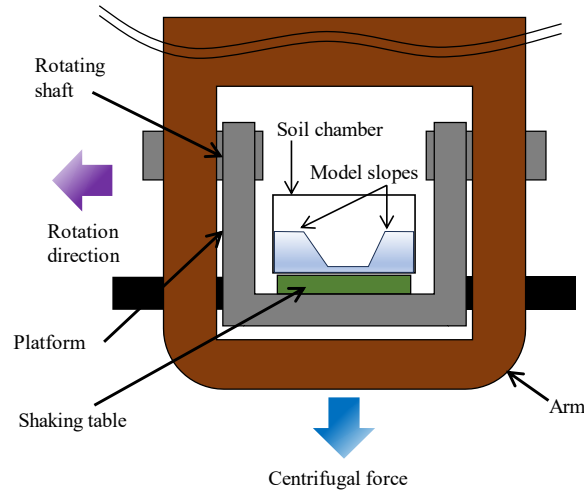


Fig. 6-3 Diagram of model slopes during centrifugal loading

6.2.2 Instrumentation

The specifications of the earth pressure gauges and laser sensors used in this chapter are the same as those described in the **Section 4.2.2**. Up to five accelerometers, type A6H-50G were placed to measure the response of the slope models during each seismic excitation.

Up to thirteen earth pressure gauges were installed within the model to monitor variations in internal earth pressure during each shaking event. Additionally, up to six laser sensors were used to measure slope displacements at specific locations during the tests.

The sensor arrangement varied slightly in each case. Most sensors were positioned at the bottom and middle heights of the model (see **Fig. 6-4** and **Fig. 6-5**). Four earth pressure sensors (P6, P7, P8, P9) were placed along the slope boundary to monitor horizontal earth pressures, while nine additional sensors (P1, P2, P3, P4, P5, P10, P11, P12, P13) were used to measure vertical earth pressures.

Slope displacements were also recorded using laser sensors. In Case 1, two laser sensors (LD1 and LD4) were used to record the slope crests' vertical displacement on both sides. In Cases 2 and 3, two additional laser sensors (LD2 and LD4) were employed to record the slope crests' horizontal displacement. In Case 4, the laser sensor measuring the vertical displacement of the left crest was removed due to space limitations. In Case 5, two additional sensors (LD3 and LD4) were installed to measure the slope surfaces' horizontal displacement at the middle height. Targets introduced in **Section 4.2.2** were also placed.

Additionally, high-speed cameras were used in each case to monitor slope deformations, providing visual evidence to complement the sensor measurements. The sensor configurations of all cases are listed in **Table 6-2**.

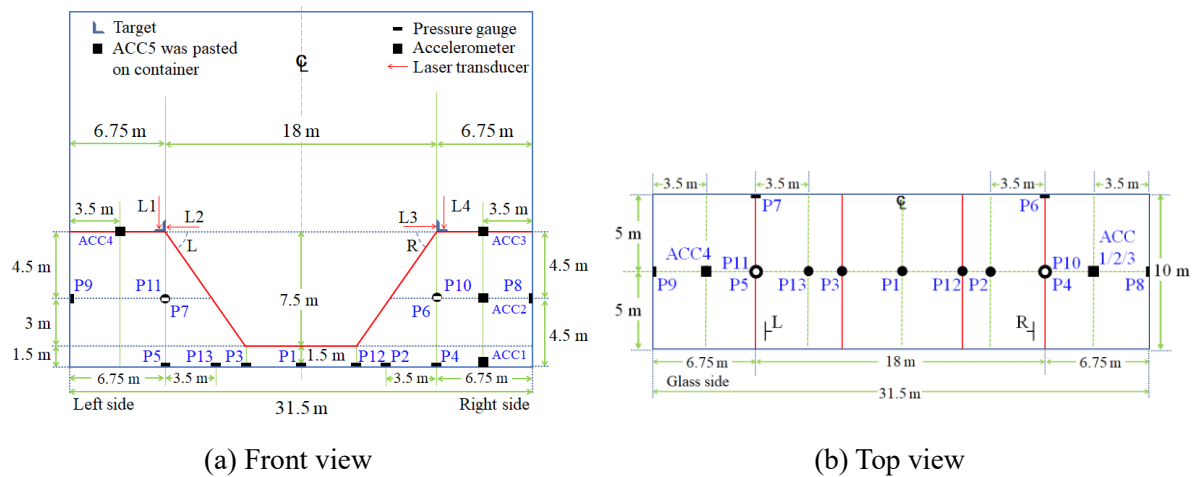


Fig. 6-4 Configuration of sensors in Case 1 to Case 4

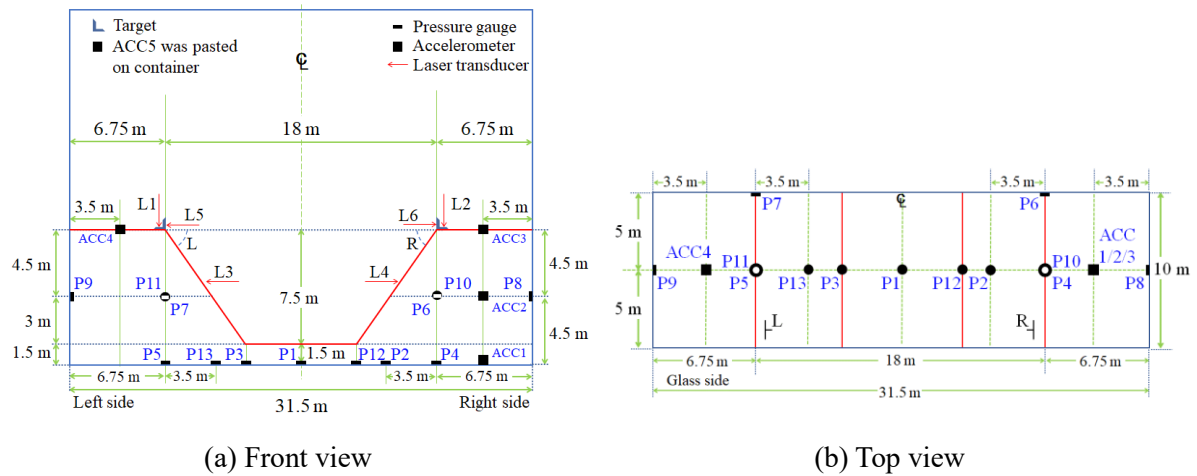


Fig. 6-5 Configuration of sensors in Case 5

Table 6-2 List of sensors for all cases

No.	Date	Earth pressure sensor													Accelerometer					Laser sensor					
		P	P	P	P	P	P	P	P	P	P	P	P	P	ACC	ACC	ACC	ACC	ACC	LD	LD	LD	LD	LD	LD
		1	2	3	4	5	6	7	8	9	10	11	12	13	1	2	3	4	5	1	2	3	4	5	6
1	211022	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
2	220412	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
3	220426	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
4	221007	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
5	221017	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned
 - The sensor was not placed

6.2.3 Test procedure

The experimental process was consistent across cases. First, dry Hiroshima sand was blended with water to achieve a desired water content of 8%. Prepared material was then divided into six equal portions and compacted into the soil chamber in six layers, each with a thickness of 3 cm, using a wooden hammer. Each layer's weight was calculated from a compaction degree ($D_c=90\%$) and the volume of each layer. Sensors were positioned when the soil was compacted to the designated heights. After forming a horizontal foundation with a height of 18 cm, unnecessary soil was excavated from the center, and two slope surfaces were shaped using a spatula. The preparation procedure is illustrated in **Fig. 6-6**. For each case, the distance from the soil chamber's wall to slope crests was kept constant (denoted by two black solid lines), while the slope angle was adjusted according to specific conditions.

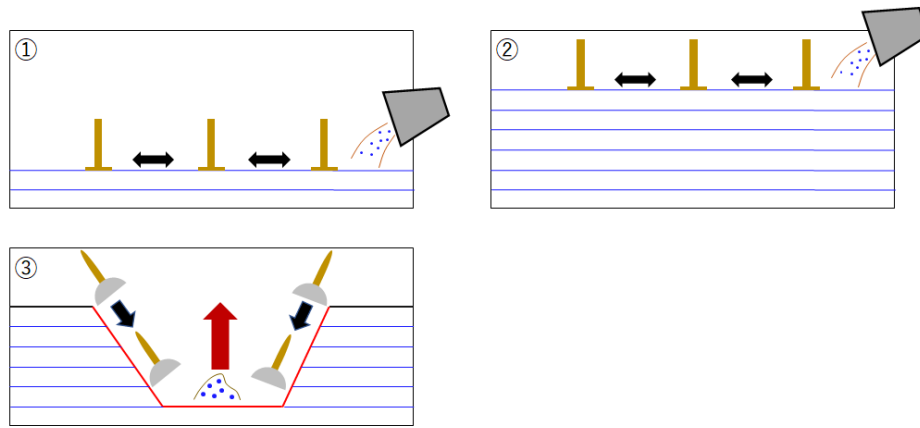


Fig. 6-6 Flow chart of the preparation procedure

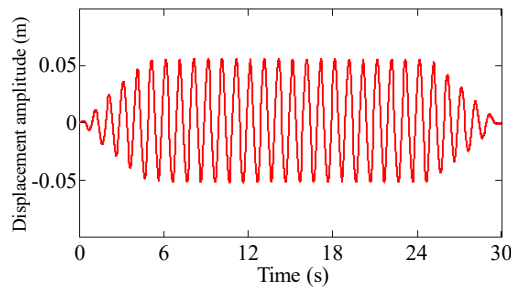


Fig. 6-7 Datum ramped sine wave (prototype scale at 50G)

The model was then installed onto the centrifuge platform for testing. Centrifugal loading was initially increased steadily from 1G to 50G. The centrifugal acceleration was then maintained for a while to stabilize sensor values. Next, seismic excitations, controlled by displacement, were applied sequentially to the model using an inflight shaking table. A ramped sine wave (see **Fig. 6-7**) having a frequency of 1 Hz, an elapsed time of 30 seconds and an amplitude of 0.05 m in prototype scale at 50G was used, with the amplitude gradually increased by magnification numbers for each subsequent wave input.

Unlike a regular sinusoidal wave, which applies maximum amplitude abruptly and can introduce sudden dynamic forces to the system, a ramped sinusoidal wave gradually increases amplitude over time, providing a smoother application of shaking forces. This approach minimizes sudden shocks to the model, enhancing realism and preventing unintended damage. The specific magnifications for every case are provided by **Table 6-1**.

6.3 Experimental results

In each case, β_L and β_R represent the slope angles of the left and right slopes, respectively. The slope sliding was monitored in detail by high-speed cameras. All sliding events occurring within one second. These events will be shown in chronological order for each case. The maximum absolute acceleration measured by an accelerometer at the model slope's bottom (ACC1) is used to indicate the intensity of each shaking input. Additionally, displacement and earth pressure data measured by installed sensors are presented below.

6.3.1.1 Case 1 ($\beta_L=55^\circ$, $\beta_R=45^\circ$)

Sliding process

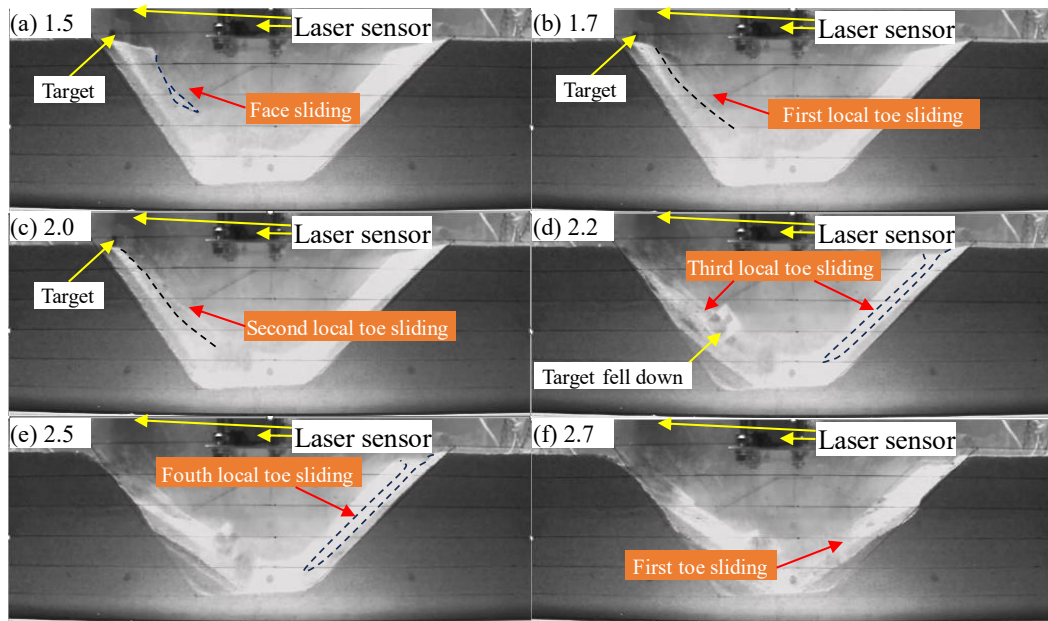


Fig. 6-8 Response of the model slopes to each shaking in Case 1: (a) face sliding on the left slope; (b) first local toe sliding on the left slope; (c) second local toe sliding on the left slope; (d) third local toe sliding on both slopes; (e) fourth local toe sliding on the right slope; (f) first toe sliding on the right

The definitions of different types of sliding are clarified as follows:

Toe Sliding: The sliding surface connects the slope crest and the toe, across the entire breadth.

Local Toe Sliding: The sliding surface connects the slope crest and the toe but only across partial

breadth.

Local Sliding: The sliding surface does not connect the slope crest and the toe.

In Case 1, eleven seismic excitations with progressively increasing amplitudes were applied. The acceleration data from ACC1 was converted into the horizontal seismic coefficient, k_h , on the prototype scale, by dividing by 50g. The corresponding k_h values at which sliding occurred are shown in **Fig. 6-9**. The model slopes remained stable after initial shakings with magnifications of 0.3, 0.5, 0.8, 1.0, and 1.2.

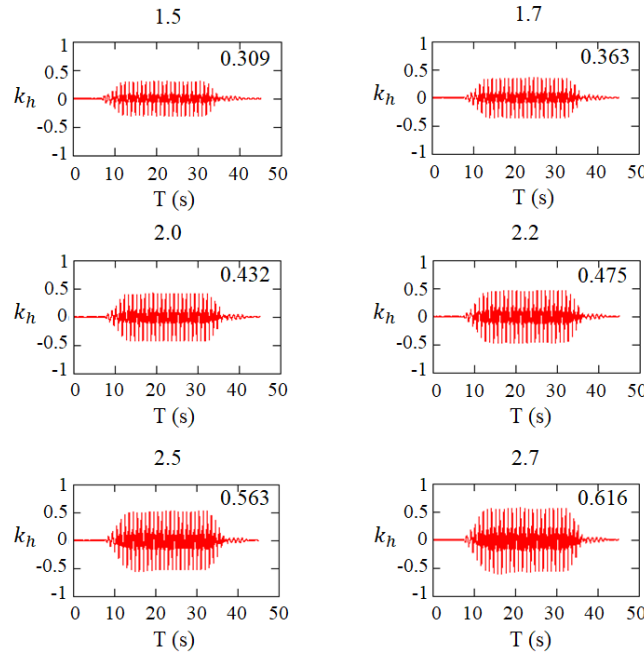


Fig. 6-9 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 1

When the magnification reached 1.5 ($k_h=0.309$), face sliding occurred near the back wall of the soil chamber on the left slope (see **Fig. 6-8 (a)**). Subsequently, the first local toe sliding occurred at the same location as the face sliding with a magnification of 1.7 ($k_h=0.363$) (see **Fig. 6-8 (b)**). The second local toe sliding further expanded the failure area on the left slope with a magnification of 2.0 ($k_h=0.432$) (see **Fig. 6-8 (c)**).

Following the shaking with a magnification of 2.2 ($k_h=0.475$), the third local toe sliding happened on both slopes (see **Fig. 6-8 (d)**). On the left slope, this sliding, located near the glass side, resulted in a complete sliding surface and caused the target to fall. On the right slope, the sliding was localized at the central part of the slope. With a magnification of 2.5 ($k_h=0.563$), the fourth local toe sliding further expanded the failure area on the right slope. Finally, a complete toe sliding occurred on the right slope after the shaking with a magnification of 2.7 ($k_h=0.616$).

Displacement

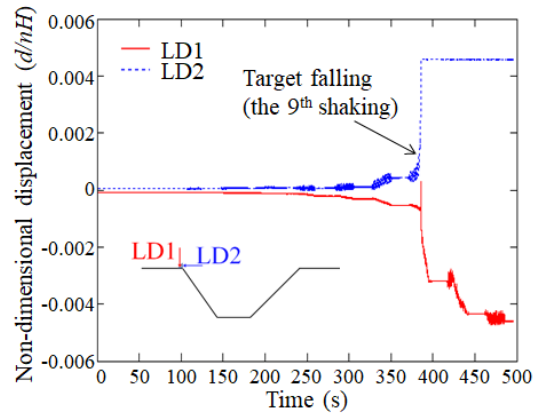


Fig. 6-10 The left slope crest' vertical and horizontal displacement of with each shaking in Case 1.

The relationship between the non-dimensional displacement, d/nH , and each shaking is presented in **Fig. 6-10**. Here, d represents real displacement measured by laser sensors, $n=50$ is the scaling factor used in centrifuge modeling to account for gravity, and H is the model' total height, which is 18 cm. Positive displacement values indicate that the target, representing displacement of the left slope crest, is moving towards the laser sensor, while a negative value indicates movement away from it.

With each shaking event, the slope crest shows further settlement and forward deformation. During the 9th shaking, when the target fell, LD2 went out of range due to the loss of the target.

Earth pressure

The change of non-dimensional earth pressure, $P/npgH$, with each shaking event is plotted in **Fig. 6-11**, where P is the earth pressure measured by sensors, and g is the gravitation acceleration, typically 9.81 m/s^2 . The earth pressure fluctuates in each shaking and its value can vary between pre- and post-shaking measurements.

The post-shaking values of sensors P1 to P5, positioned at the bottom of the model, gradually increase with each shaking, while other pressure sensors display less noticeable changes. Sensors P6 to P9 show minimal difference between pre- and post-shaking values throughout the test. Sensors P10 and P11 have a similar characteristic. At first several shakings, their post-shaking value is bigger than their pre-shaking values. However, after specific shakings, one or more post-shaking values drop below the pre-shaking values. This reduction occurs in the 9th shaking for P10 and the 7th shaking for P11.

As previously discussed, the initial sliding events occurred during the 9th shaking on the right slope and the 6th on the left slope, indicating that slope sliding corresponds with the observed decrease in post-shaking earth pressure.

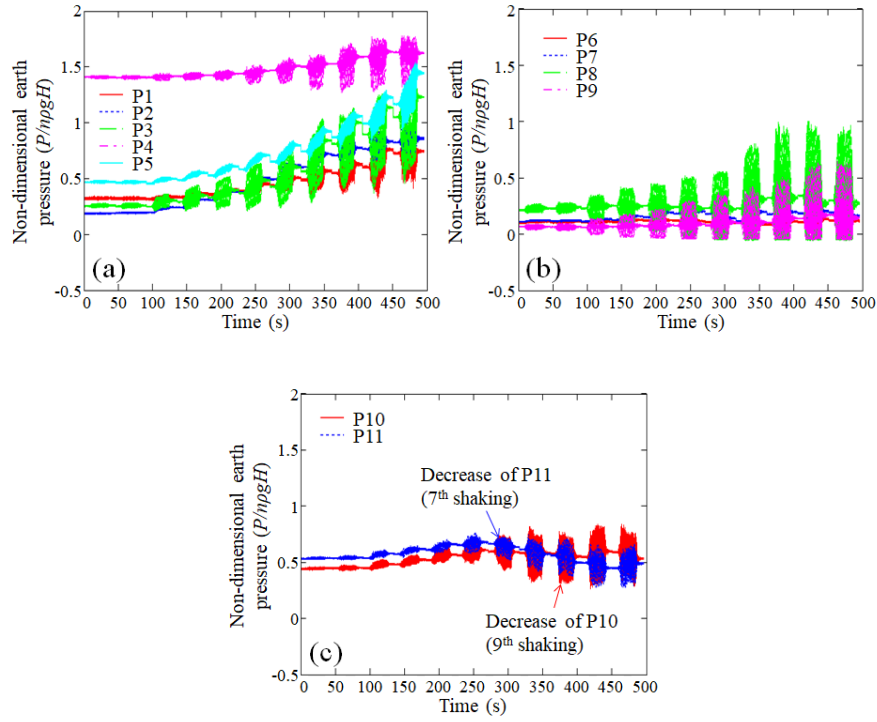


Fig. 6-11 Measured earth pressure with each shaking in Case 1: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 6-12 illustrates the measured accelerations at various locations of the model slope during each shaking event. ACCT records the output wave from the shaking table. The acceleration values have been scaled to the prototype scale k_h by dividing by 50g. A noticeable seismic amplification effect is evident, as accelerations increase with elevation.

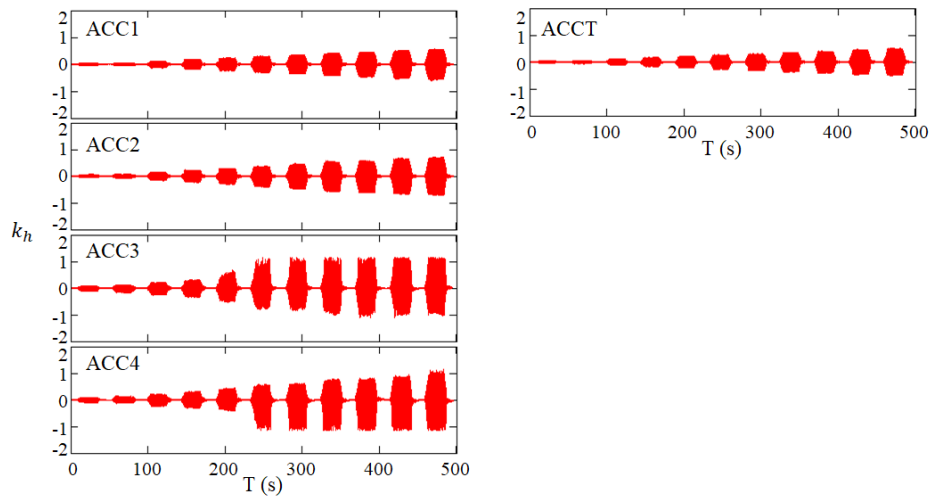


Fig. 6-12 Measured acceleration with each shaking in Case 1

6.3.1.2 Case 2 ($\beta_L=55^\circ$, $\beta_R=45^\circ$)

Sliding process

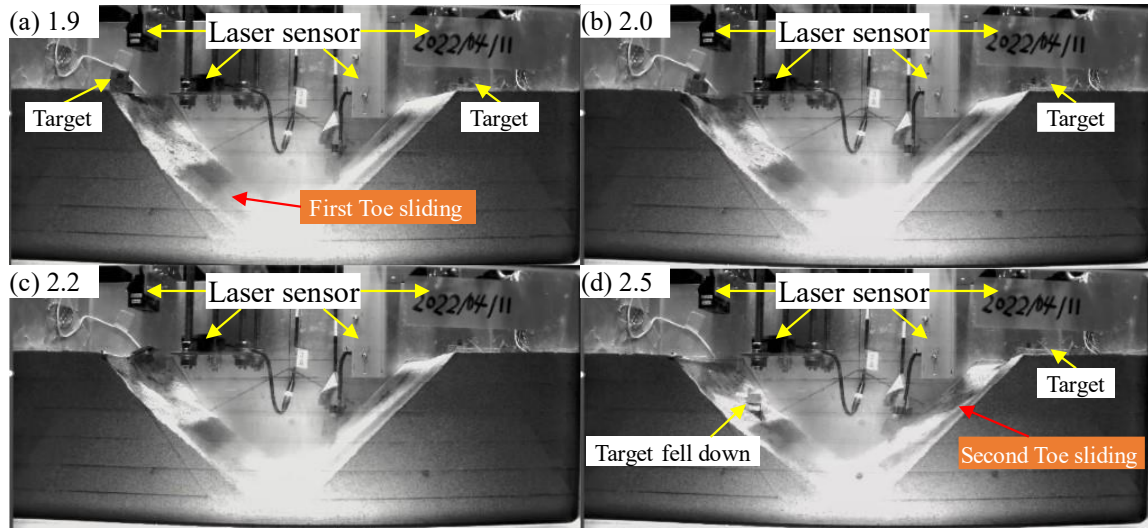


Fig. 6-13 Reaction of the model slopes to each shaking in Case 2: (a) First toe sliding on the left slope; (b) slope state after the second shaking; (c) slope state after the third shaking; (d) second toe sliding on the right slope

In Case 2, eight seismic excitations with progressively increasing amplitudes were applied. The corresponding k_h values at which sliding happened are shown in **Fig. 6-14**. After the fourth shaking, with a magnification of 1.9 ($k_h=0.345$), the first toe sliding occurred on the left slope. Following the next two shakings (2.0 and 2.2 magnifications), the model remained stable. After the final shaking, with a magnification of 2.5 ($k_h=0.481$), the second toe sliding was observed at the right slope.

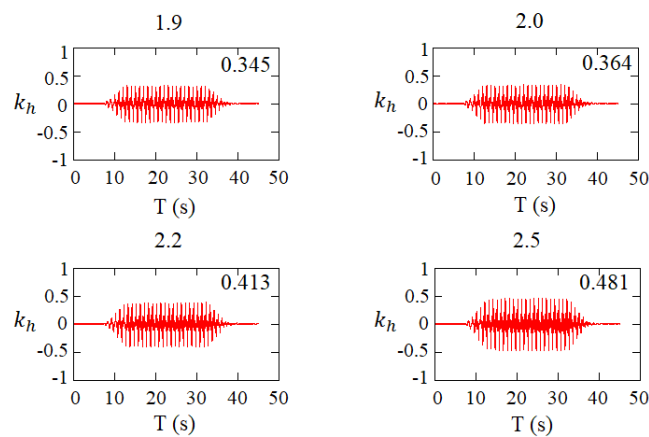


Fig. 6-14 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 2

Displacement

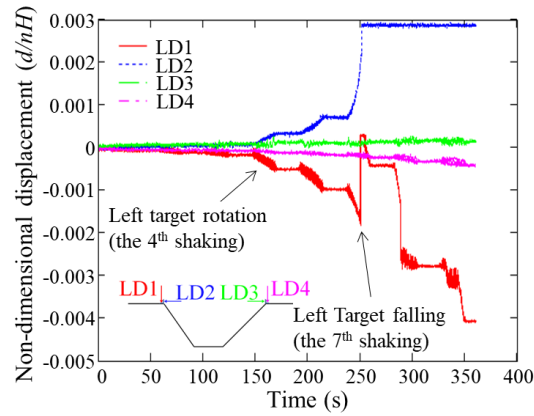
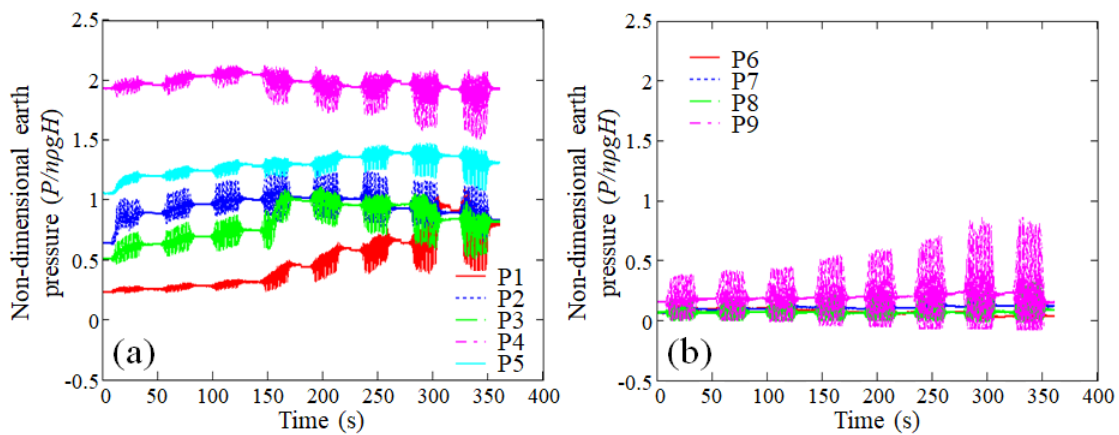


Fig. 6-15 Vertical and horizontal displacement of both slope crests with each shaking in Case 2

The symbols in **Fig. 6-15** have the same meanings as those in **Fig. 6-10**. In Case 2, an additional target was placed on the right slope crest, which remained stable throughout all shakings, resulting in no abrupt changes in LD3 and LD4. The left target rotated during the fourth shaking, resulting in an rise in LD2 and a reduction in LD1. During the seventh shaking, the left target fell, causing LD1 to decrease and LD2 to go out of range.

Earth pressure

Fig. 6-16 presents the variation of non-dimensional earth pressure during each shaking in Case 2, with symbols corresponding to those in **Fig. 6-11**.



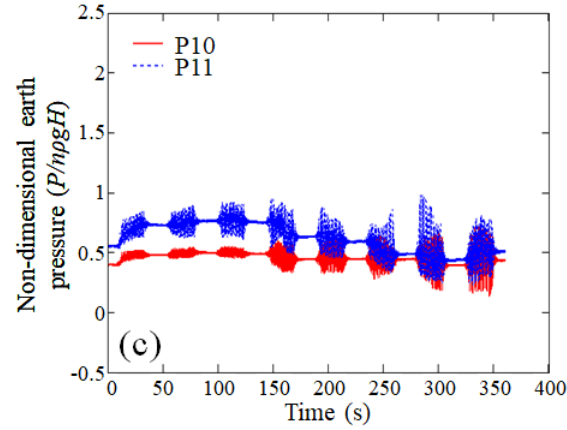


Fig. 6-16 Measured earth pressure with each shaking in Case 2: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

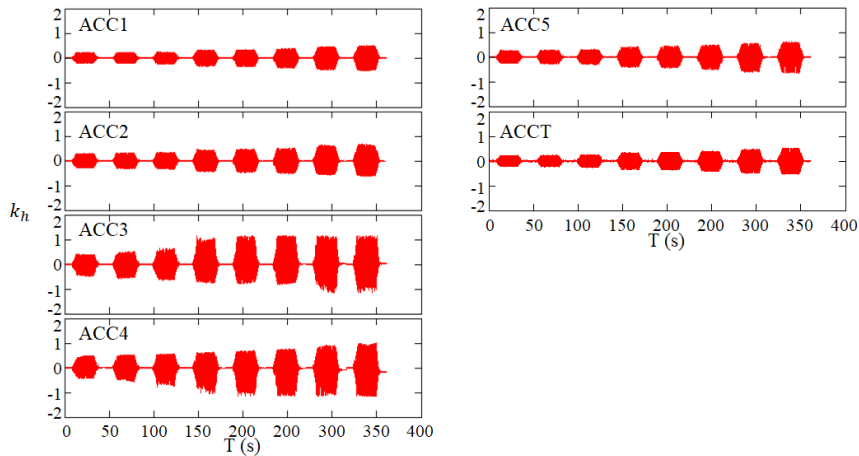


Fig. 6-17 Measured acceleration with each shaking in Case 2

Fig. 6-17 illustrates the measured accelerations at various locations on the model slope during each shaking. ACCT records the output wave from the shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. A seismic coefficient effect is observed, as acceleration increases with elevation.

6.3.1.3 Case 3 ($\beta_L=55^\circ$, $\beta_R=45^\circ$)

Sliding process

In Case 3, two seismic excitations were applied, with their corresponding k_h values at the onset of sliding shown in **Fig. 6-19**. Before shaking, two cracks had already initiated in the plasticine due to deformation from the 50G acceleration. During the first shaking, with a magnification of 1.9 ($k_h=0.381$)

both slopes remained stable despite further deformation of the plasticine. However, in the second shaking, with a magnification of 2.5 ($k_h=0.526$), both slopes failed simultaneously due to toe sliding. The plasticine on the left slope bent significantly under the thrust force of the soil debris accumulating behind it.

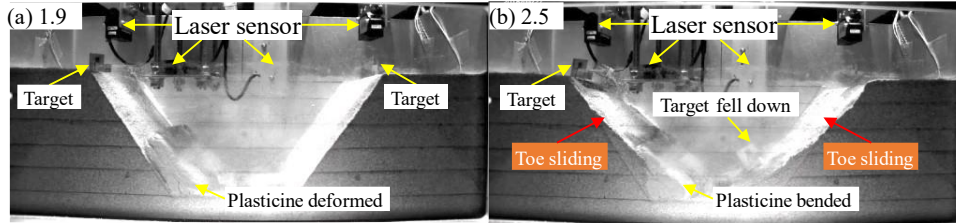


Fig. 6-18 Reaction of the model slopes to each shaking in Case 3: (a) deformation of the plasticine in the first shaking; (b) toe sliding on both slopes

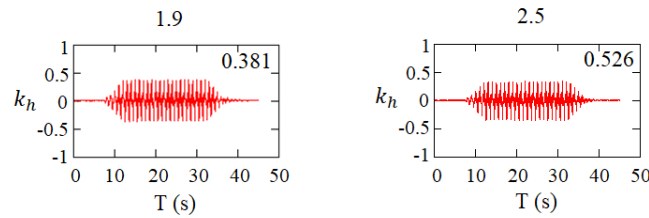


Fig. 6-19 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 3

Displacement

The symbols in **Fig. 6-20** have the same meanings as those in **Fig. 6-10**, showing the variation of non-dimensional displacement with each shaking in Case 3. During the second shaking, the increase in LD2 and decrease in LD1 resulted from the rotation of the target on the left crest. Additionally, LD3 went out of range, and LD4 decreased as the right target fell.

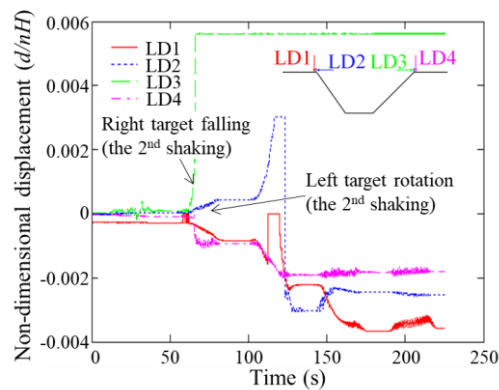


Fig. 6-20 Vertical and horizontal displacement of both slope crests with each shaking in Case 3

Earth pressure

Fig. 6-21 presents the variation of non-dimensional earth pressure during each shaking in Case 3, where the symbols can refer to those in **Fig. 6-11**. The values of P3 and P5 are larger than those of the other sensors on the bottom due to the presence of plasticine at the left slope toe.

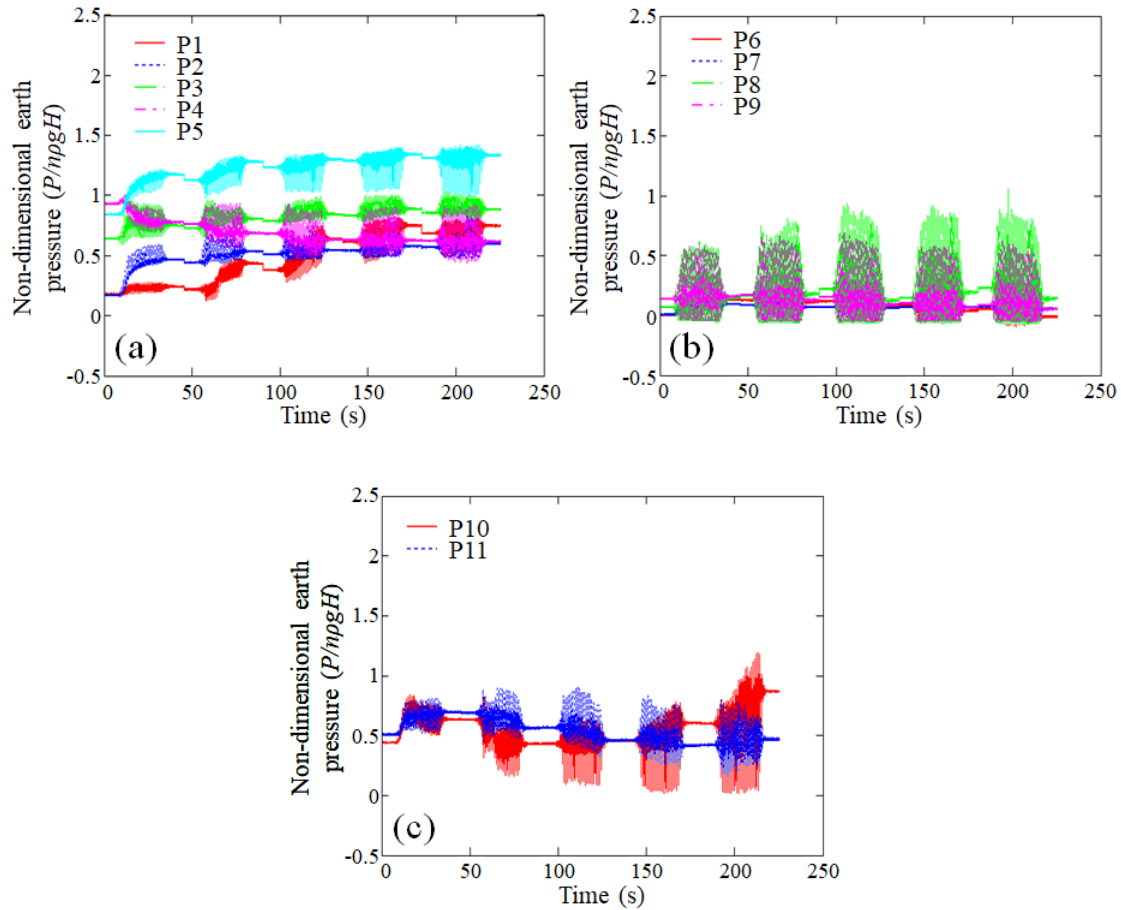


Fig. 6-21 Measure earth pressure with each shaking in Case 3: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 6-22 illustrates the measured accelerations at various locations on the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

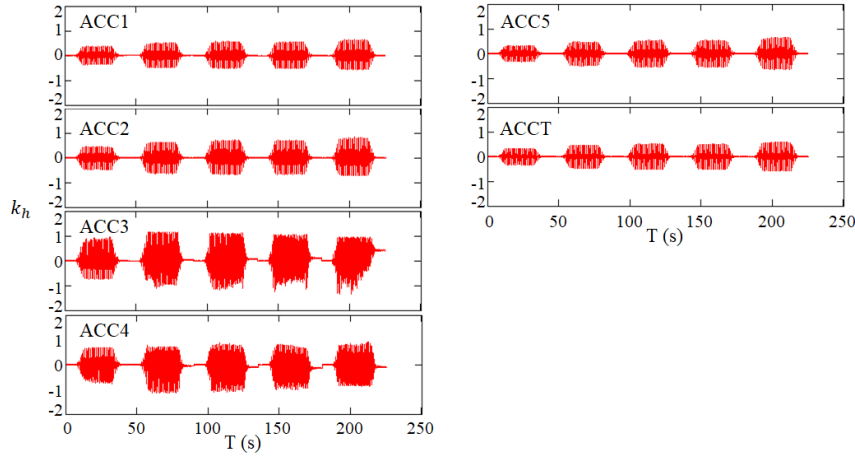


Fig. 6-22 Measured acceleration with each shaking in Case 3

6.3.1.4 Case 4 ($\beta_L=55^\circ$, $\beta_R=55^\circ$)

Sliding process

In Case 4, two seismic excitations were applied. Their corresponding k_h values at the time of sliding are shown in **Fig. 6-24**. The left slope, with a box on the toe, was not the primary focus of this case. The right slope failed during the first shaking with a magnification of 1.9 ($k_h=0.383$). From the bending points of noodles, the sliding surface was observed to be a slightly curved plane. The second shaking, with a magnification of 2.5 ($k_h=0.535$), aggravated the failure of the right slope.

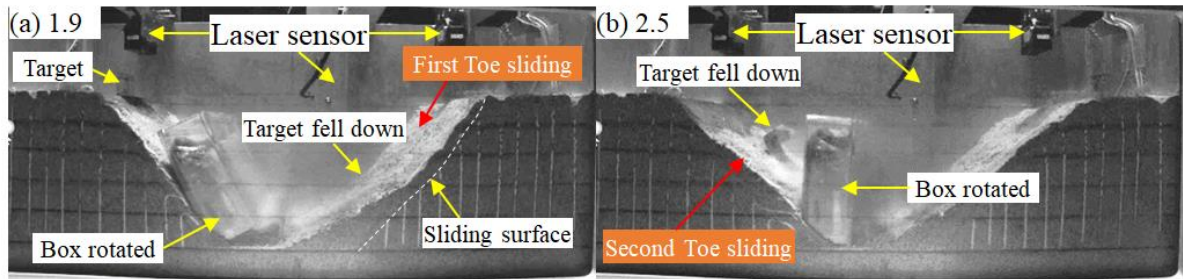


Fig. 6-23 Reaction of the model slopes to each shaking in Case 4: (a) first toe sliding on the right slope; (b) second toe sliding on the left slope

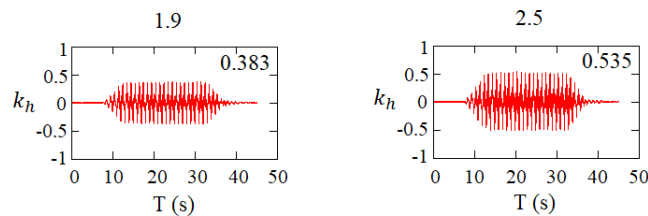


Fig. 6-24 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 4

Displacement

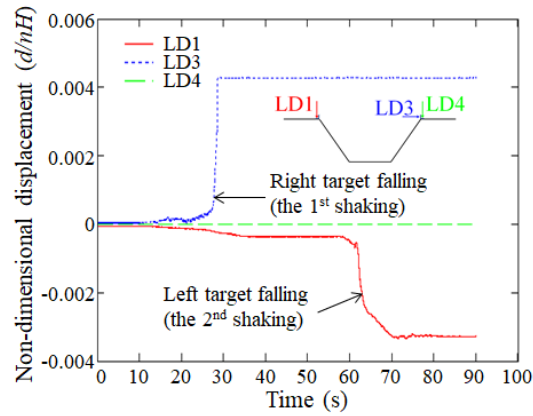
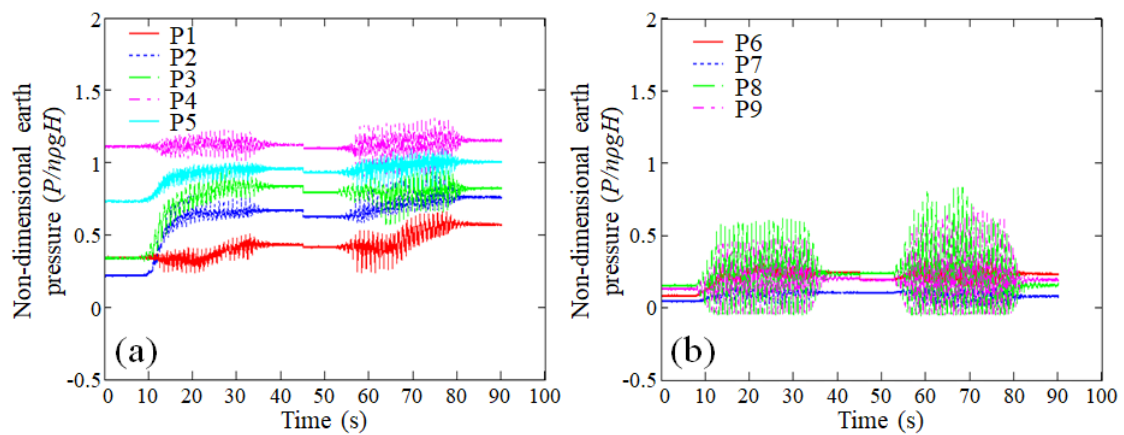


Fig. 6-25 Vertical displacement of left slope crest and vertical and horizontal displacement of right slope crests with each shaking in Case 4

The symbols in **Fig. 6-25** have the same meanings as those in **Fig. 6-10**, illustrating the variation of non-dimensional displacement with each shaking in Case 4. The value of LD4 remained unchanged throughout the test due to a malfunction. LD3 went out of range because the right target fell during the first shaking. During the second shaking, the value of LD1 decreased due to toe sliding on the left slope.

Earth pressure

Fig. 6-26 shows variation of non-dimensional earth pressure with each shaking in Case 4, where the symbols can refer to those in **Fig. 6-11**.



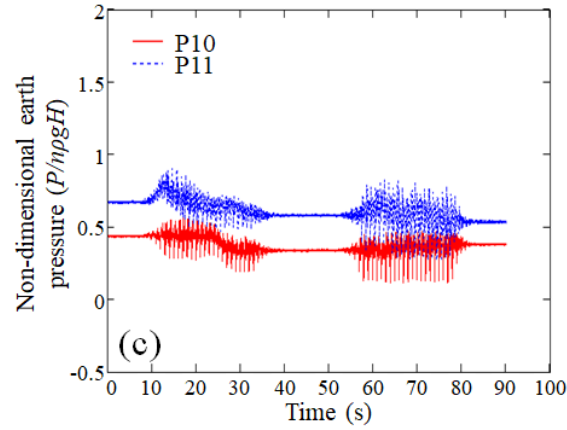


Fig. 6-26 Measure earth pressure with each shaking in Case 4: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 6-27 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of the shaking table. The acceleration values have been converted to the prototype scale, k_h , by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

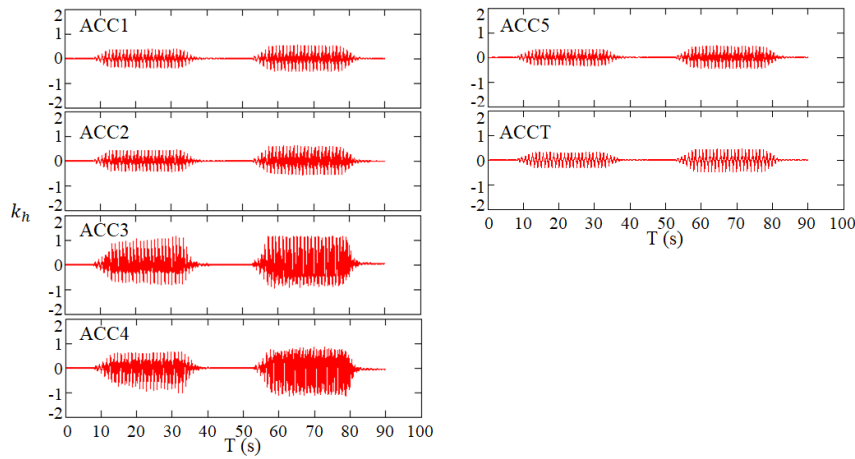


Fig. 6-27 Measured acceleration with each shaking in Case 4

6.3.1.5 Case 5 ($\beta_L=55^\circ$, $\beta_R=55^\circ$)

Sliding process

In Case 5, two seismic excitations with increasing amplitudes were applied. Their corresponding k_h values when sliding occurred are shown in **Fig. 6-29**. After the first shaking, with a magnification of 1.9 ($k_h=0.360$), a local toe sliding happened on both slopes near the glass side. During the second

shaking, with a magnification of 2.5 ($k_h=0.501$), both slopes failed with toe sliding.

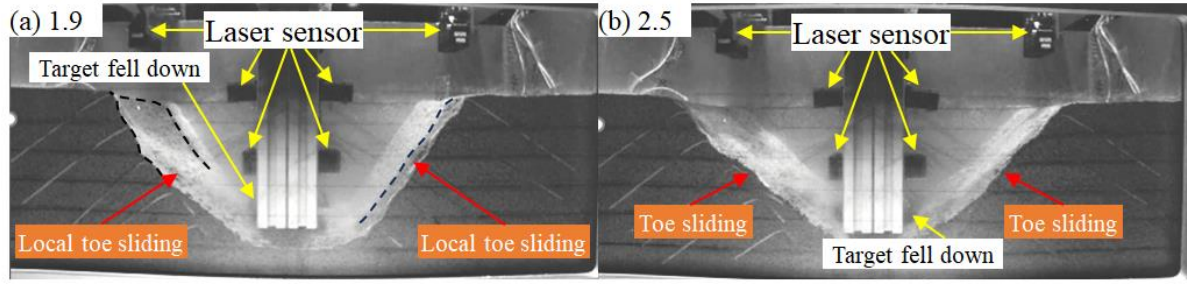


Fig. 6-28 Reaction of the model slopes to each shaking in Case 5: (a) local sliding on both slopes; (b) toe sliding on both slopes

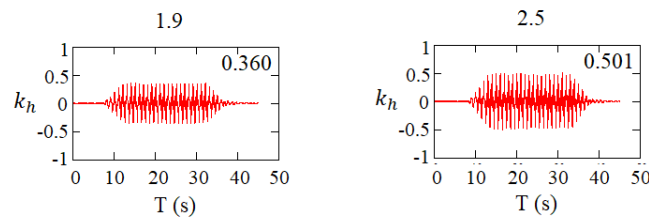


Fig. 6-29 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 5

Displacement

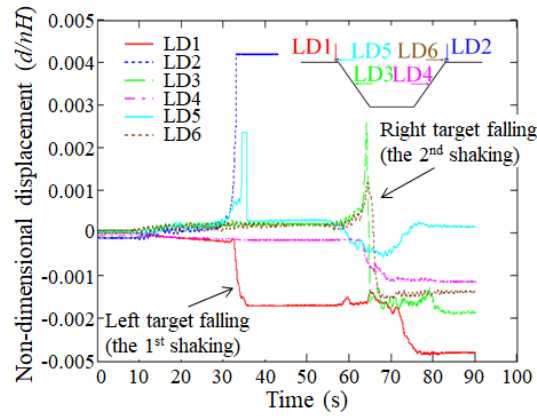


Fig. 6-30 Vertical and horizontal displacement of crests and horizontal displacement of surfaces of both slopes with each shaking in Case 5

The symbols in **Fig. 6-30** have the same meanings as those in **Fig. 6-10**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 5. In this case, LD3 and LD4 were installed to monitor the displacement of both slope surfaces.

Earth pressure

Fig. 6-31 shows variation of non-dimensional earth pressure with each shaking in Case 5, where the symbols can refer to those in **Fig. 6-11**.

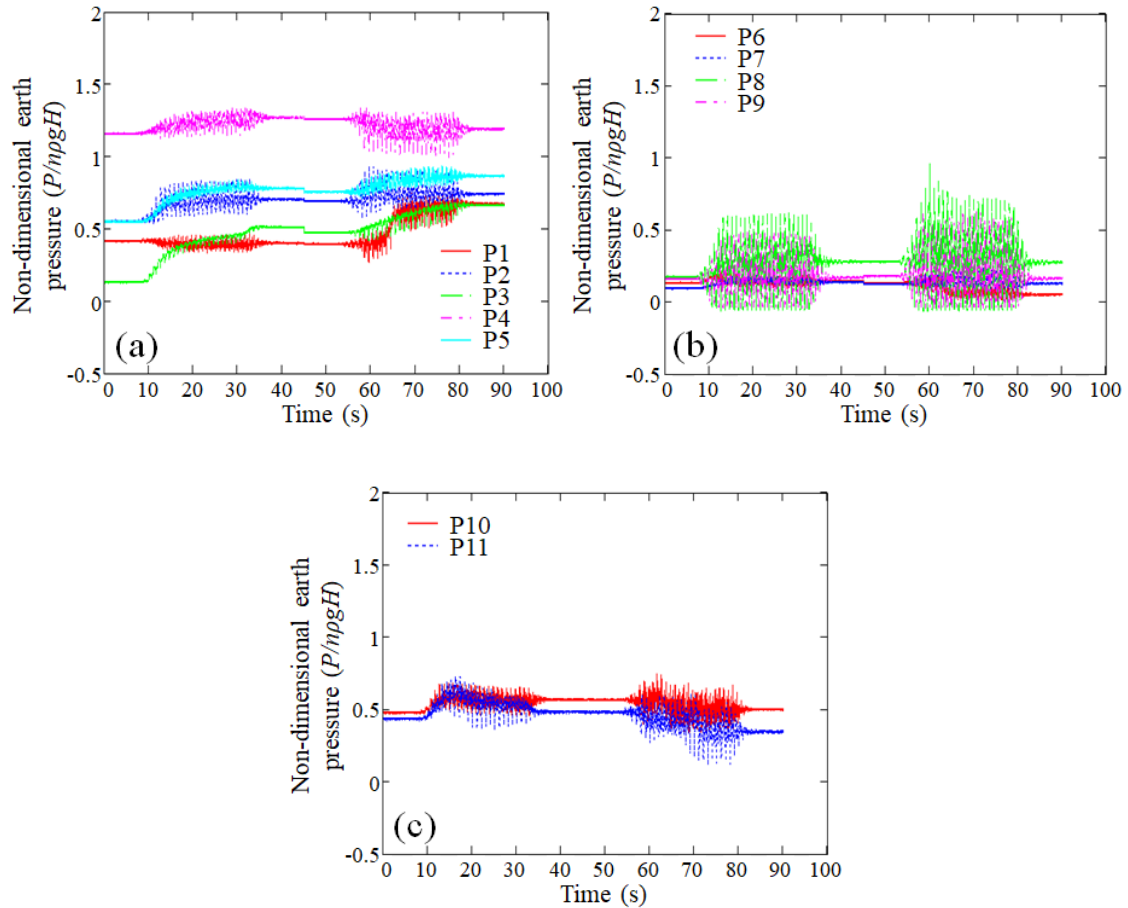


Fig. 6-31 Measure earth pressure with each shaking in Case 5: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 6-32 illustrates the measured accelerations at various locations of the model slope during each shaking in Case 5. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

The experimental outcomes of all cases are listed in **Table 6-3**. After the testing, the water content in the model slope was sampled from three distinct heights from the bottom of the model. The seismic coefficients at the onset of the first toe sliding or local toe sliding are also presented.

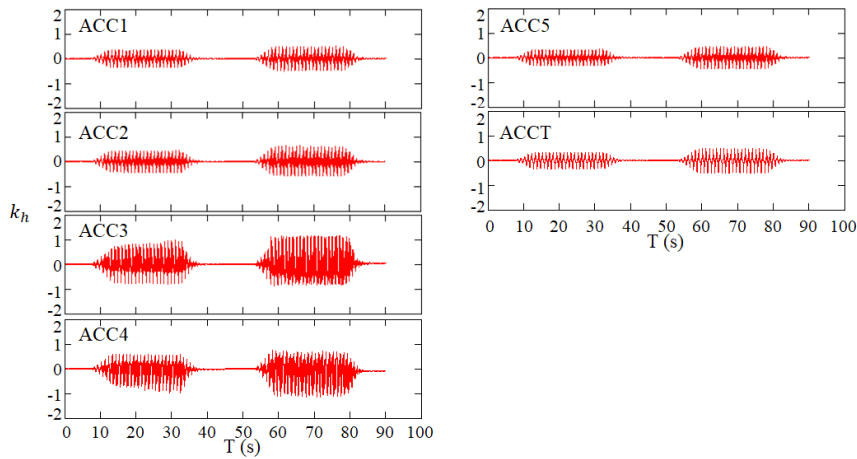


Fig. 6-32 Measured acceleration with each shaking in Case 5

Table 6-3 Experimental results of uniform slopes

No.	Date	Final water content (%)			Sliding seismic coefficient	
		Bottom (0-2 cm)	Middle (8-10 cm)	Top (16-18 cm)	Left	Right
1	211022	Δ	Δ	Δ	0.363 (1.7)	0.475 (2.2)
2	220412	11.7	6.8	6.6	0.345 (1.9)	0.481 (2.5)
3	220426	9.6	6.5	6.2	0.526 (2.5)	0.526 (2.5)
4	221007	8.6	7.4	5.9	0.535 (2.5)	0.383 (1.9)
5	221017	10.1	6.4	6	0.360 (1.9)	0.360 (1.9)

6.4 Discussion

6.4.1 Stability and failure modes of slopes in the test

The experiments indicate that the slope inclination is crucial to slope dynamic stability. Reducing the slope inclination not only increases the seismic coefficient required to trigger slope failure but also reduces the amount of sliding soil.

Compared to the static cases in the previous two chapters, the dynamic cases exhibited more frequent toe sliding across the entire slope surface, whereas in the static cases, sliding typically initiated near the wall. This difference may be attributed to the higher compaction degree in dynamic cases, which likely rendered the model slope more homogeneous.

The left slope in Case 1 demonstrated greater stability than Case 2, despite having the same inclination. This increased stability is likely due to the application of additional low-intensity seismic waves in Case 1, which may have helped to strengthen the model slopes by densifying them.

6.4.2 Overburden pressure at the bottom of the model

In each case, up to seven earth pressure sensors (P1 to P5, P12, and P13) were placed along the longitudinal centerline at the model bottom to record changes in overburden pressure during ground motions. Sensor P1 was positioned under the model's center. Sensors P2 and P13 were placed below the slope surfaces, while P3 and P12 were located directly below the slope toes. Sensors P4 and P5 were placed vertically below the slope crests.

Fig. 6-33 compares the normalized overburden height through back calculation, before and after seismic loadings (50G), with the normalized actual overburden height. The earth pressure increased after the shaking. According to Nguyen and Pipatpongsa (2020) as well as Wanjala et al. (2020), when the energy from the seismic loading is sufficiently large to overcome deformation thresholds, stress histories induced by the construction process, the structure of grain contacts, the variations in local density and the initial friction on the lateral confinements are erased.

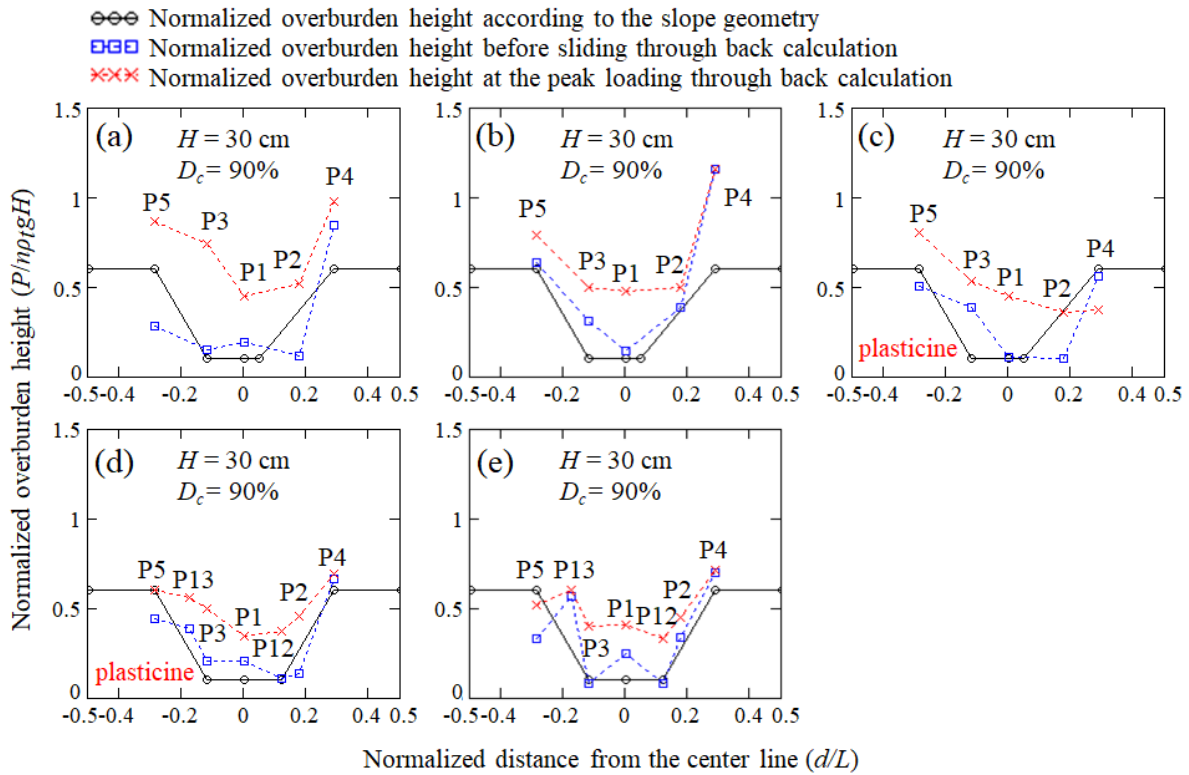


Fig. 6-33 Normalized overburden height at bottom of the model before and after seismic loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5

6.4.3 Stress-strain relationship before failure

In Case 1, three accelerometers, labeled ACC1, ACC2, and ACC3, were placed vertically on the right-hand slope (see **Fig. 6-34**). The slope is classified into an upper and a lower part. Shear stresses at $3/4 H$ and $1/4 H$ of the model height selected to represent the shear stress of each part, respectively, to account for variations in shear stress at two heights.

Calculation methods of shear stress and strain is shown in **Fig. 6-34**, using **Eqs. (6-12), (6-13), (6-14)** and **(6-15)**. Herein, γ_1 is the shear strain between ACC1 and ACC2 and γ_2 is the shear strain between ACC2 and ACC3. The shear strain is assumed to be the same between each two accelerometers where u_1 and u_2 are displacements integrated from acceleration values a_1 and a_2 , respectively. The shear strain is computed from the differential displacement using adjacent accelerometers (Yasuyuki & Osamu, 1990). The value of acceleration is thought to be linearly distributed between two accelerometers. The dash line divides the slope into three blocks. Block A is used to calculate τ_2 and block B can be used to calculate τ_1 . B is the assumed width of the virtual soil column and can be removed in the derivation process.

$$\gamma_1 = \frac{u_2 - u_1}{h} \quad (6-12)$$

$$\gamma_2 = \frac{u_3 - u_2}{h} \quad (6-13)$$

$$\tau_1 = \tau_2 + \frac{m_B \cdot a_B}{B} = \rho_t \left(\frac{9h}{8} a_2 + \frac{3h}{8} a_3 \right) \quad (6-14)$$

$$\tau_2 = \frac{m_A \cdot a_A}{B} = \rho_t \left(\frac{h}{8} a_2 + \frac{3h}{8} a_3 \right) \quad (6-15)$$

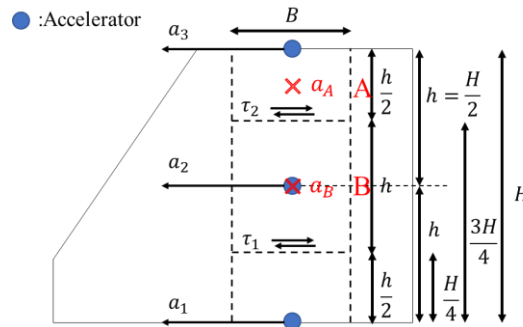


Fig. 6-34 Calculation method of shear stress and strain

Firstly, the 5th shaking before sliding on both slopes is selected to show the distribution of acceleration. As the recorded waves have noises that may affect the results of integration for displacements, they need filtering at the first step. In order to decide the filtering method, the FFT (fast Fourier transform) is used. The single-sided amplitude spectrums for those three accelerometers are plotted by FFT as below (see **Fig. 6-35 (a)**). For all recorded accelerations, there are two predominant frequencies, one is at 1 Hz and another is at 3 Hz in prototype scale. The wave having a frequency of 1 Hz is main frequency and that with 3 Hz is caused by resonance or reflection. Those two waves are superposed to achieve the maximum amplitude. Here, those waves having a frequency lower than 1 Hz regarded as noise are removed by using high-pass filter. The waves after being filtered are shown in **Fig. 6-35 (b)**. It can be observed clearly that the acceleration ascends with height, showing an amplification of the seismic wave.

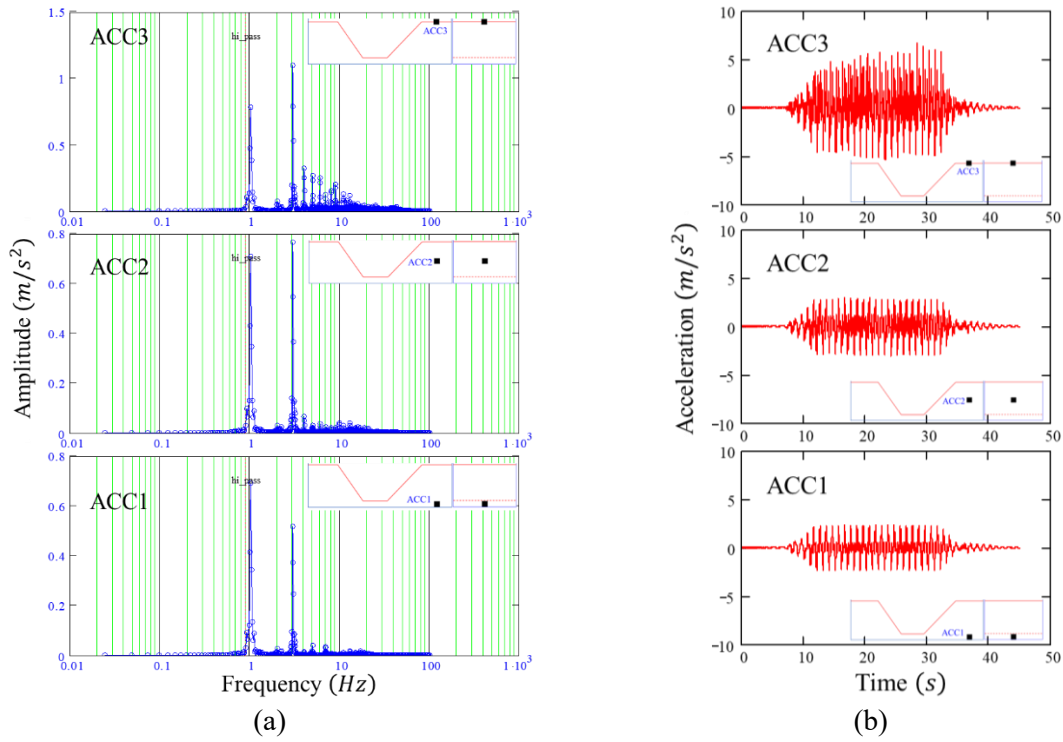


Fig. 6-35 Process of measured acceleration at different heights: (a) single-sided amplitude spectrum; (b) Acceleration along the height (Prototype scale)

The stress and strain relationship are calculated using the method according to the above-mentioned formulae. Then, their relationships are plotted as follows (see **Fig. 6-36**). Since two component waves exist with different frequencies, the shape of the hysteresis loops is not like a spindle as usual where, $\max(|\tau_1|)=37.2$ kPa, $\max(|\gamma_1|)=6.9 \times 10^{-4}$, $\max(|\tau_2|)=19.8$ kPa, $\max(|\gamma_2|)=1.9 \times 10^{-3}$. Although $|\tau_1| > |\tau_2|$, the lower part of the slope has a bigger shear stress, while absolute value of shear strain for the lower part is smaller compared to the upper part. This means that the shear modulus of the lower section is bigger. The upper part's strain magnitude reaches the range of large strains.

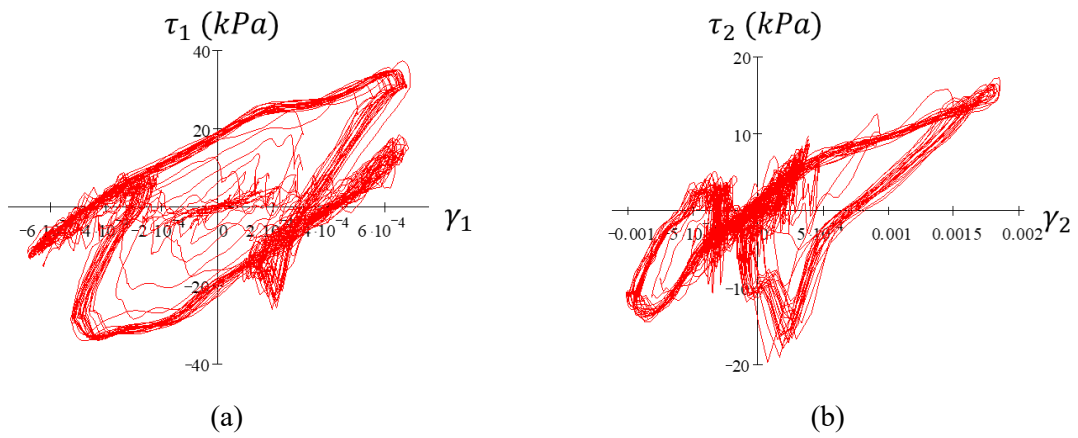


Fig. 6-36 Stress-strain relationship on the 5th shaking: (a) lower section of the slope; (b) higher section of the slope

6.4.4 Seismic amplification with increasing magnitude of ground motions

Due to the layered compaction during construction, the model slope exhibits a stratified structure, which influences the seismic amplification phenomenon observed in the acceleration time history (see **Section 6.3**). To clarify this effect and emphasize its relationship with the input wave magnitude, the dimensionless acceleration amplification factor (AAF) is used to represent slope response, where the AAF is defined as the ratio of the peak response acceleration at any point within the slope to the peak response acceleration on the shaking table (Xin et al., 2024). The absolute maximum values from each acceleration recording curve was identified as peak values.

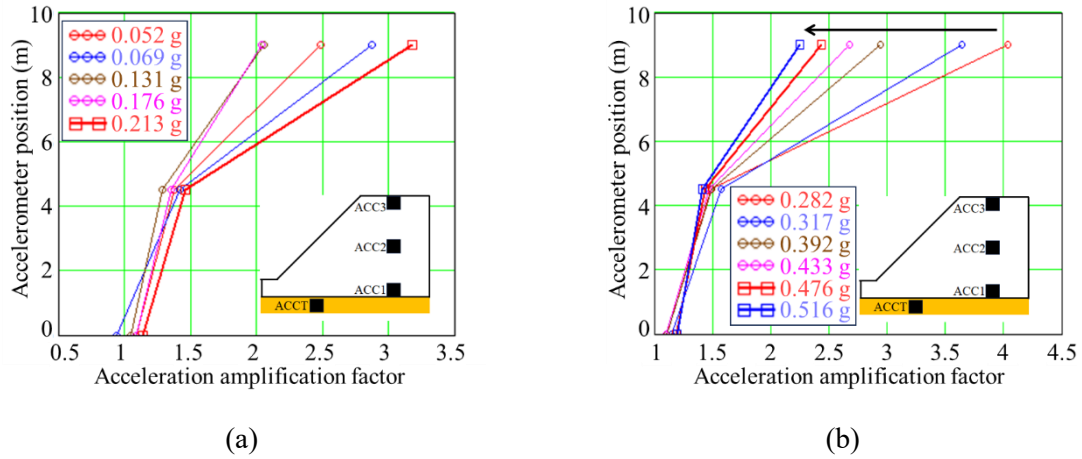


Fig. 6-37 AAF of the uniform slope in Case 1: (a) the first five shakings; (b) the last six shakings

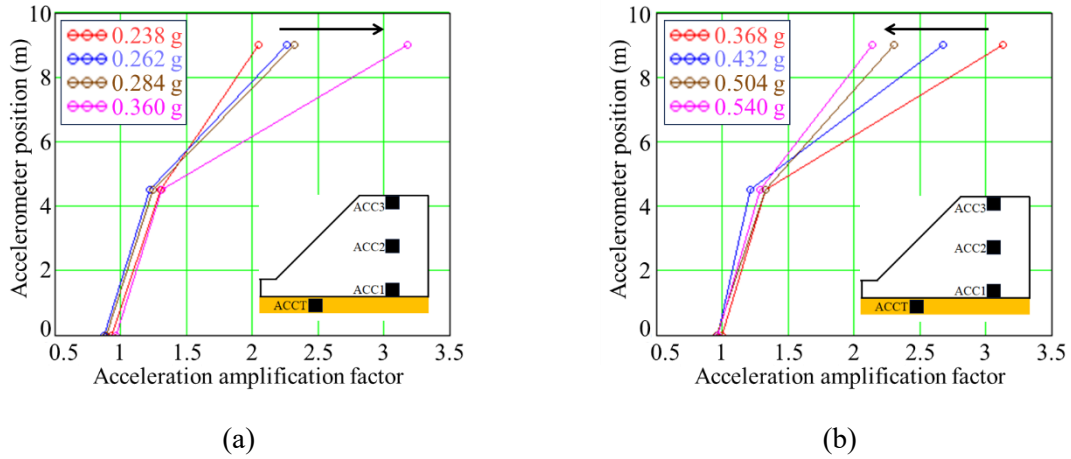


Fig. 6-38 AAF of the uniform slope in Case 2: (a) the first four shakings; (b) the last four shakings

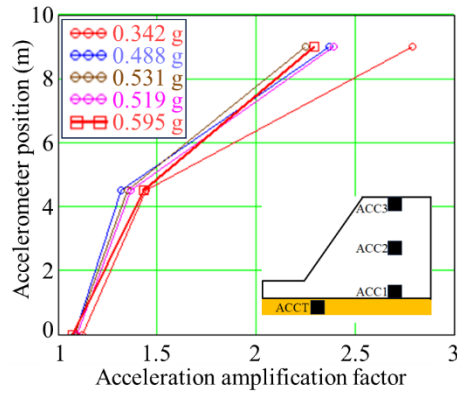


Fig. 6-39 AAF of the uniform slope in Case 3

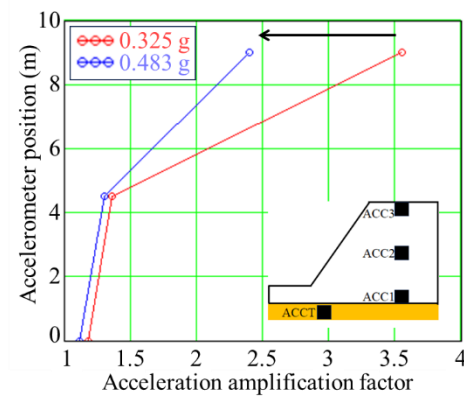


Fig. 6-40 AAF of the uniform slope in Case 4

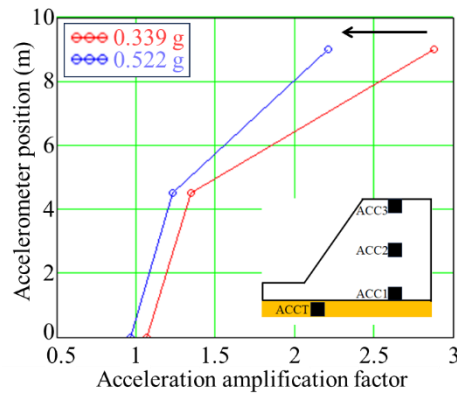


Fig. 6-41 AAF of the uniform slope in Case 5

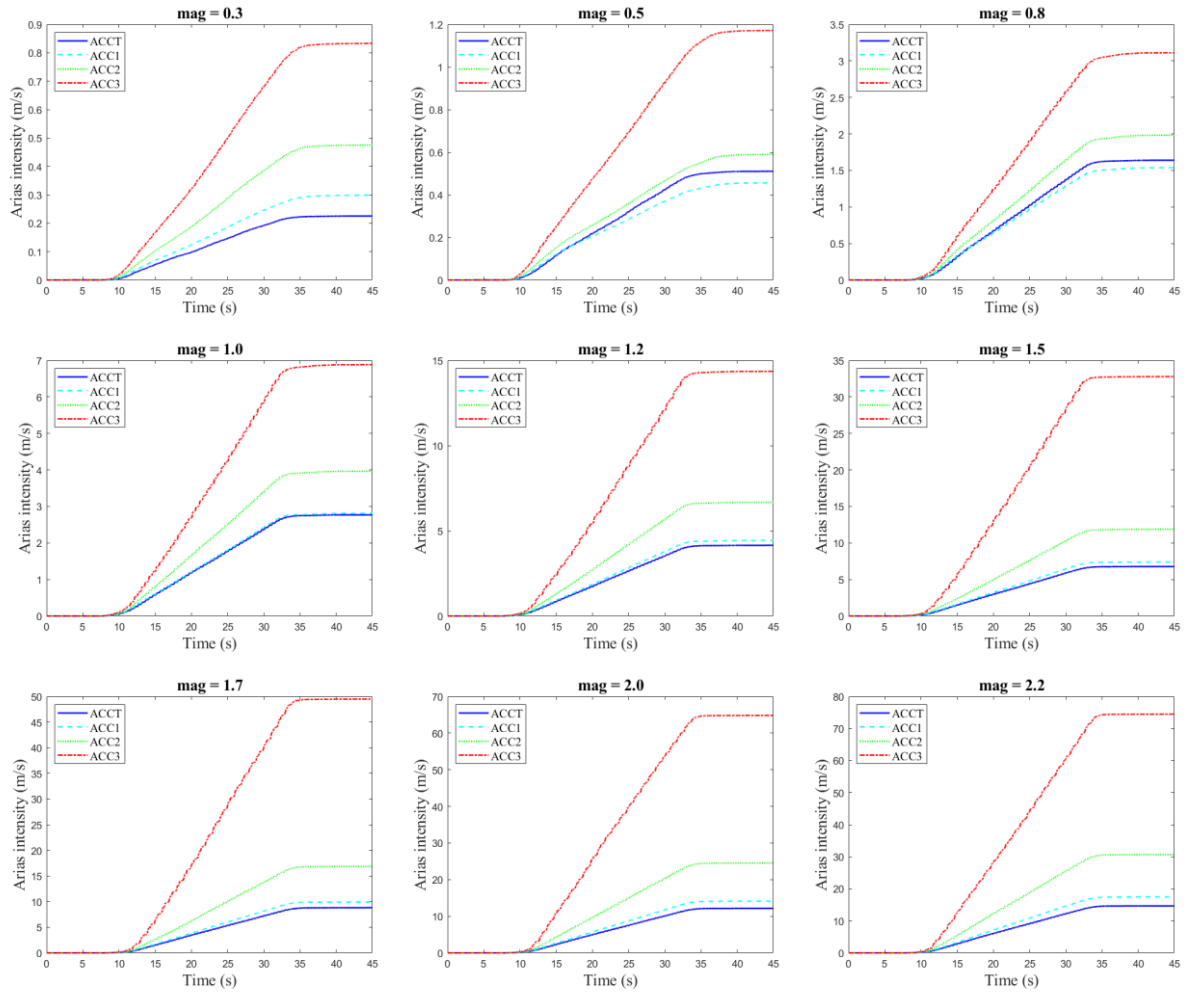
Fig. 6-37 to Fig. 6-41 illustrate elevation amplification effect across the five cases. Generally, the peak acceleration measured by ACC1, placed within the model slope at the bottom of the soil chamber, is higher compared to the input peak value from the shaking table. Acceleration increases nonlinearly from the bottom to the top, with the amplification level varying according to the input motion's magnitude. At the model's middle height (ACC2), changes in acceleration are minor, but at the top

(ACC3), the amplification variation is significant. When the input wave magnitude exceeds a threshold of 0.3 g, the AAF at the model top tends to decrease, as presented in **Fig. 6-37 (b)**, **Fig. 6-38 (b)**, **Fig. 6-40** and **Fig. 6-41**. Conversely, for input magnitudes below 0.3 g, the AAF may increase, as in **Fig. 6-38 (a)**. Strong ground motions can induce internal failure within the model, reducing its integrity, natural frequency, and increasing damping effects. Consequently, less energy is transmitted to the model's top, causing the seismic amplification effect to diminish during intense shaking.

While acceleration provides direct observational data on seismic motion, Arias intensity, which was originally proposed by Arias in 1970, serves as a broader indicator of seismic motion intensity. It accounts for additional factors, including amplitude, duration, and frequency, offering a more comprehensive representation of the energy released during seismic events (Xin et al., 2024).

Arias intensity, denoted as I_a , is calculated using **Eq. (6-16)**, where T_d represents the seismic motion's duration and $a(t)$ denotes the recorded acceleration time history.

$$I_a = \frac{\pi}{2g} \int_0^{T_d} [a(t)]^2 dt \quad (6-16)$$



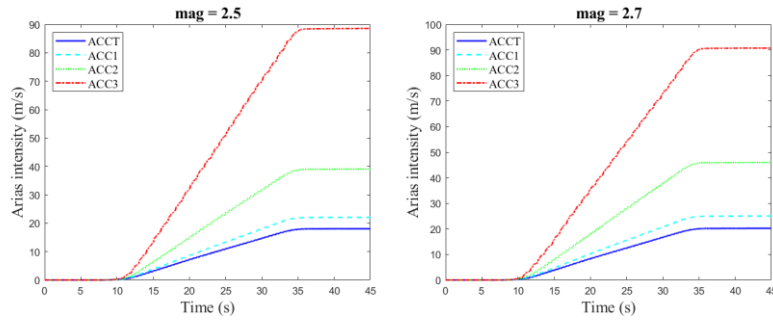


Fig. 6-42 Arias intensity evolution at various heights in Case 1

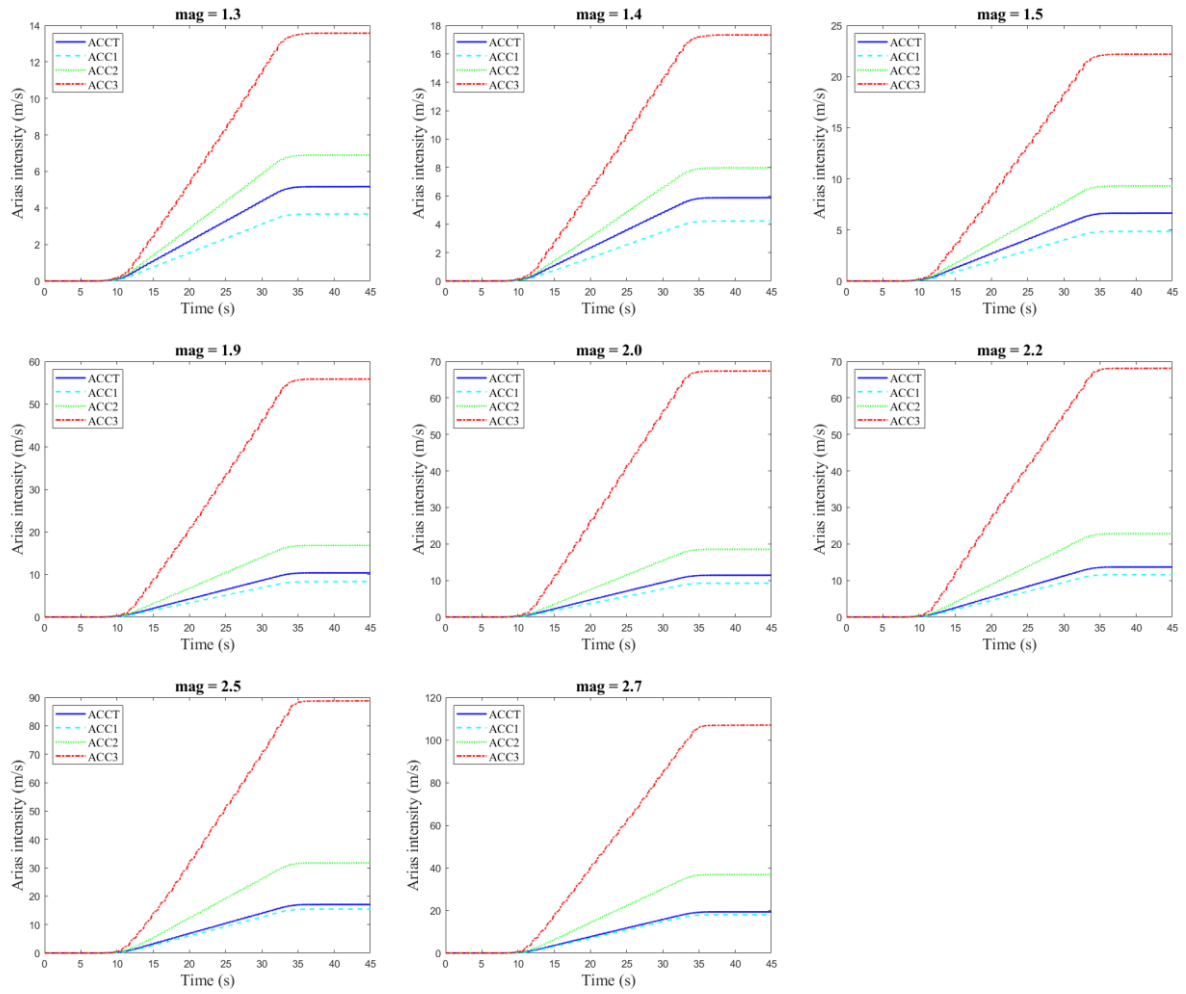


Fig. 6-43 Arias intensity evolution at various heights in Case 2

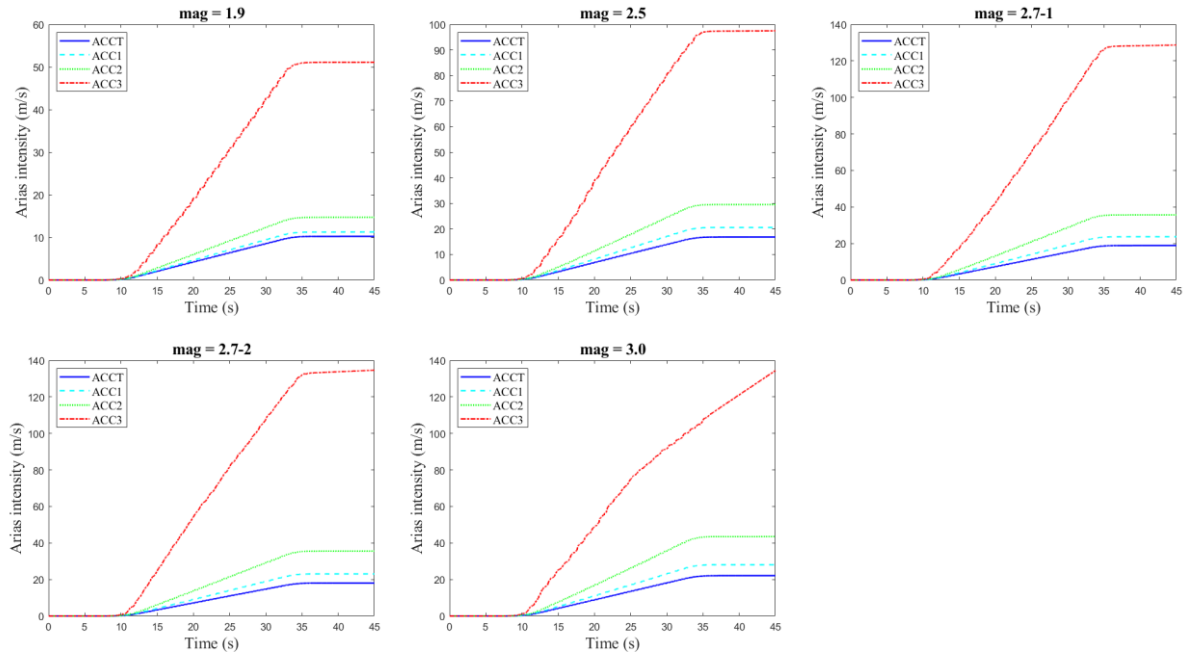


Fig. 6-44 Arias intensity evolution at various heights in Case 3

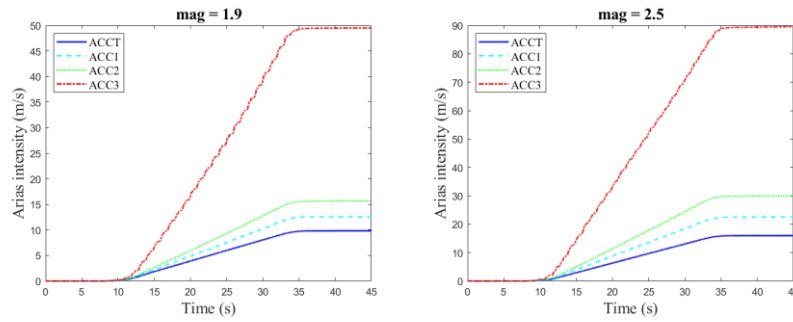


Fig. 6-45 Arias intensity evolution at various heights in Case 4

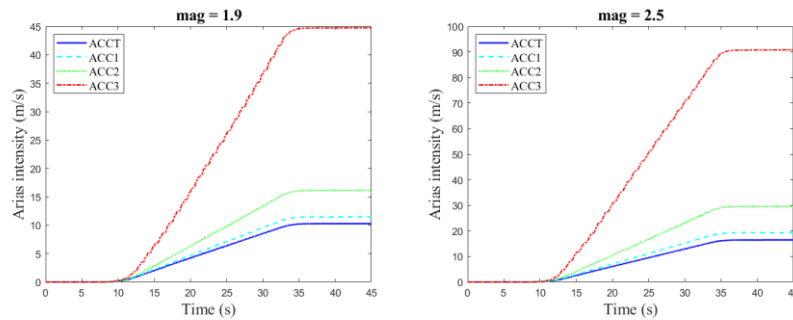


Fig. 6-46 Arias intensity evolution at various heights in Case 5

Fig. 6-42 to Fig. 6-46 illustrate the change of Arias intensity at different heights with time, capturing the energy release of seismic motions impacting the models. The ramped sine wave produces an almost linear energy release. A distinct increase of Arias intensity with elevation is found. The value calculated by ACC3 is significantly larger than the others. This trend results from the positive correlation between acceleration and Arias intensity over duration, indicating the elevation amplification effect (EAE). In

Case 3, final steady stage of Arias intensity at ACC3 is missing (see **Fig. 6-44**) due to movement of the accelerometer during intense shaking.

6.4.5 Comparison between the experiments and the theory

The centrifugal tests recorded the ranges of horizontal seismic coefficients that triggered slope sliding for slopes with varying inclinations. The correlation between the predicted k_h and the slope inclination (β) can be determined using **Eq. (6-10)** by setting the scaling factor (n_f) to 50.

The experimental outcomes are summarized and compared with the theories in **Fig. 6-47**. In the figure, the plus symbols represent the seismic coefficient at the onset of sliding, while the dots indicate the preceding seismic coefficient. Since the seismic coefficient did not increase continuously during the experiments, the actual critical seismic coefficient likely lies between these values. All test results align with the upper 80% confidence boundary, suggesting the theoretical prediction using the mean peak shear strength parameters serves as a conservative lower boundary.

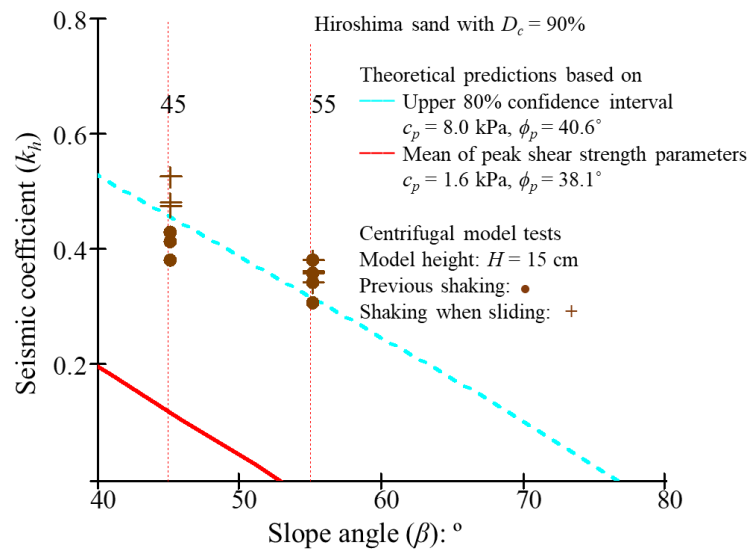


Fig. 6-47 Centrifugal model tests and theoretical predictions from the mean and the upper range of shear properties for Hiroshima sand with water content of 6%-10%

6.4.6 Variation of the slope state with ground motions

As mentioned in **section 4.1.3**, the reduction ratio, r_d , expressed by **Eq. (4-45)** can represent the slope state, ranging from the active state to a passive state. During the test, the model slopes deformed under centrifugal loading, resulting in varying slope states. For each slope, two r_d values can be calculated, producing a total of four r_d values. For example, in the right slope of Case 1, a vertical earth pressure sensor (P10, located on the center line of the model) and two horizontal earth pressure sensors (P6 on the back wall, P8 on the right wall) were positioned 9 cm above the bottom of the soil chamber, hence, by combining P10 with P6 and P8 yields two r_d values (Tao & Pipatpongsa, 2023).

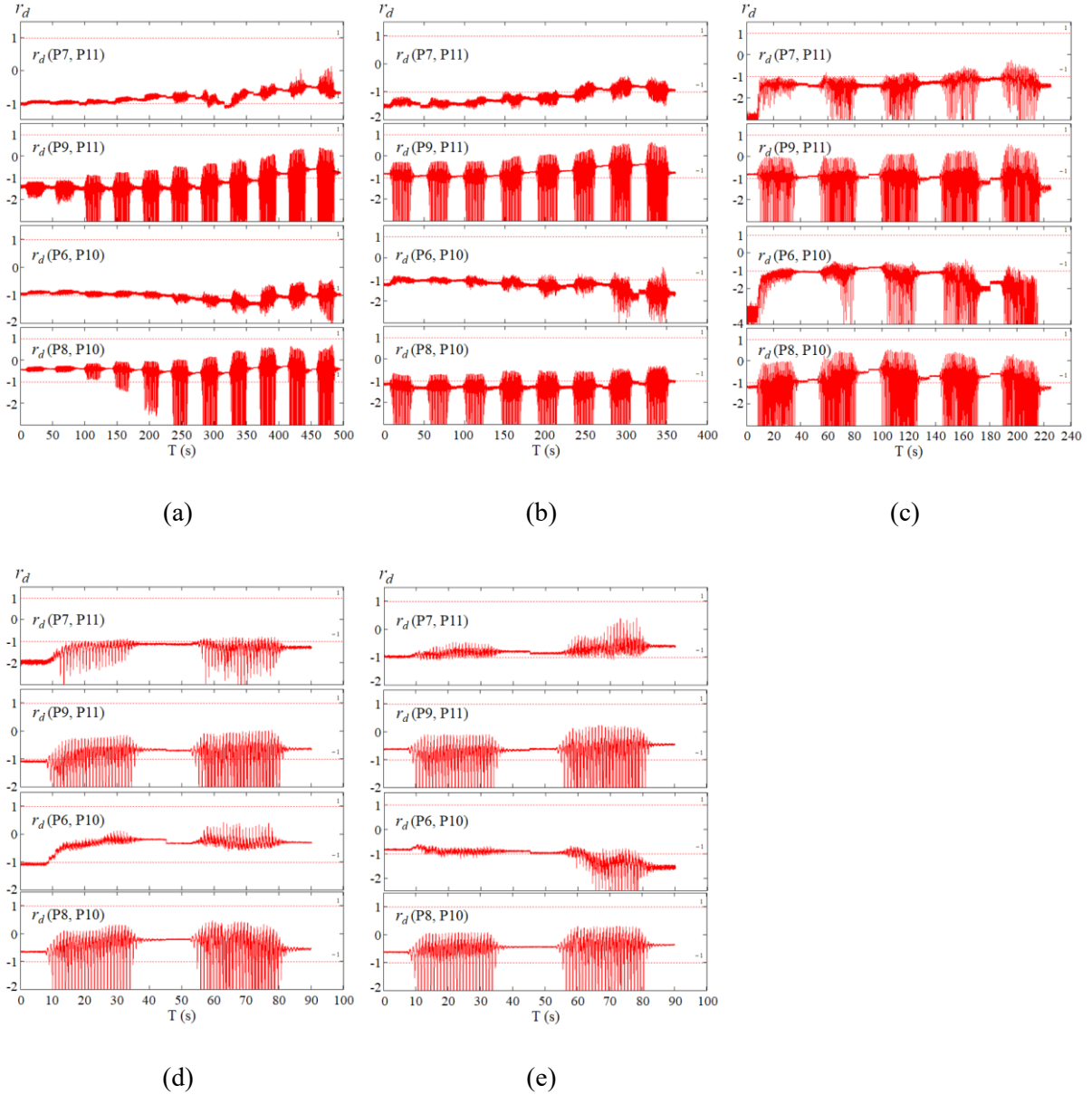


Fig. 6-48 Variation of reduction ratio with ground motions: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5

Fig. 6-48 shows the reduction ratio (r_d)'s variation with each shaking. Upon reaching 50G, before any seismic loading, the model was already in a limit active state with r_d around -1. The initial shaking with a low intensity typically increased the value of r_d , while stronger seismic loading caused it to decrease. Additionally, r_d values calculated from the pressure sensors on the side wall (P6 and P7) showed less pronounced changes than those from sensors on the left and right walls (P8 and P9) indicating that earth pressure along the shaking direction fluctuated more significantly. Overall, r_d could reach positive values during shaking, indicating that the slope could temporarily enter a passive state.

6.4.7 Impact of lateral resistance considering 3D conditions

The lateral resistance increases the uniform slope's seismic stability, as presented in **Fig. 6-49 (a)**. When r_d equals -1, the 3D condition reduces to a 2D condition. The stabilization effect becomes more pronounced when the slope is in a passive state, with r_d values greater than 0. The prediction boundary aligns closely with the upper 80% confidence interval for cohesion (see **Fig. 6-47**) when r_d equals 0.7.

Based on the measured r_d in **Section 6.4.6**, a value of zero is used to reflect the impact of lateral resistance, as shown in **Fig. 6-49 (b)**. All experimental results can fall within the predicted range if both the confidence interval and lateral support are included.

In the 3D theory, the breadth of the model (B) must first be determined (see **Eq. (6-11)**). In centrifugal modeling, this corresponds to the breadth of the soil chamber. For engineering applications, the equation is typically used for back-analysis after a failure event to estimate site material properties or seasonal variations in the water table (Pipatpongsa et al., 2022). In certain scenarios, the slope's breadth is readily apparent, such as the slope described in Brideau and Stead (2012), which features lateral kinematic constraints. Similarly, for an undercut slope, the breadth of the upper slope is equal to the width of the excavation (Mohammad et al., 2013).

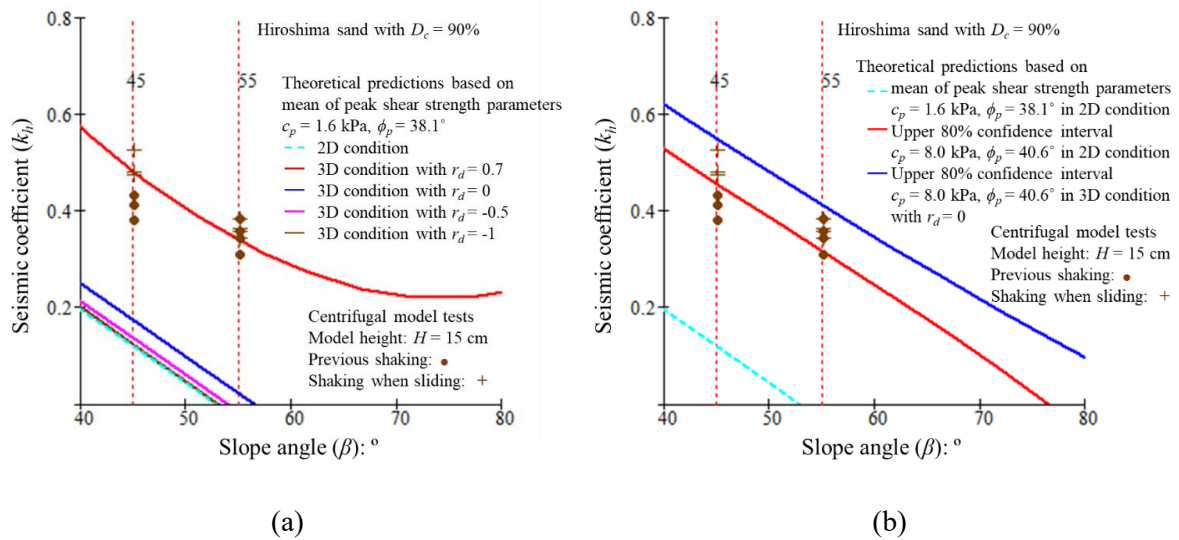


Fig. 6-49 3D analysis considering lateral resistance: (a) centrifugal model tests and theoretical predictions under 3D conditions considering the lateral resistance; (b) Centrifugal model tests and theoretical predictions considering the lateral resistance based on the observed r_d

6.5 Conclusions

This chapter studied the seismic stability of uniform steep slopes using pseudo-static methods and dynamic centrifugal model tests. The shallow sliding was triggered by a series of input waves to determine the threshold magnitude for a certain slope inclination. The failure modes and seismic reaction of the models were observed. The main findings are listed as follows.

(1) Due to a higher compaction degree, the model slopes in the dynamic cases exhibited better homogenous than those in the static cases in the previous chapters.

(2) Seismic waves with magnitude lower than the threshold value can increase the slope stability by densifying the slope through shaking. This can be proved by the measured vertical earth pressures in the slope, which increases gradually with low-intensity shakings. The change of earth pressure after each shaking in the model bottom is much more obvious than that in other places. This reflects the sufficient seismic energy can overcome deformation thresholds, thus, stress histories are erased.

(3) With three vertically aligned accelerometers, the elevation amplification of the seismic wave can be observed. By using acceleration amplification factor (AAF) and Arias intensity, it was noticed that the reaction acceleration in the slope increased non-linearly with the height and became pronounced in the slope crest. This nonlinearity increased first, after the seismic intensity surpassed the threshold, it decreased. Additionally, the lower half of the slope shows a bigger shear modulus than the upper part. It can be caused by the model construction process as the lower part was more compacted.

(4) By using pseudo-static method, the critical seismic coefficient can be calculated considering a confidence interval for the shear strength. However, the predicted value forms a lower boundary.

(5) During the shaking, the reduction factor of the slope fluctuates around -1, whose amplitude increases with the magnitude of the shaking. The slope can reach the passive state sometimes, so lateral resistance contributes to the seismic slope stability under dynamic conditions.

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Chapter 7 Evaluation of dynamic stability of benched slopes by centrifugal model tests

The seismic resistance and failure patterns of benched slopes are investigated in this chapter through dynamic centrifugal model tests. The pseudo-static Culmann's method derived in **Chapter 6** is applied to assess the stability variations resulting from toe counterweight and weight reduction. The boundary effect is also evaluated using the reduction factor (r_d) as described in **Section 7.1**.

Two groups of centrifugal model tests with benched slope models are conducted to verify the proposed theories. The experimental scheme is outlined in **Section 7.2**. The first group includes seven cases with variations in slope angles (β), first bench width (B_W) and first bench height (B_H). The second group consists of two cases with different numbers of benches, maintaining the same weight of excavated area. The material used, Hiroshima sand, was characterized in **Chapter 3**.

In **Section 7.3**, the recorded horizontal seismic coefficients at the onset of shallow sliding and the observed variations in failure modes among model slopes with different configurations are presented. Additionally, data from installed sensors is provided to illustrate the variation in earth pressure and slope displacement during shaking.

The experimental results are then compared with the pseudo-static Culmann's method under 2D conditions in **Section 7.4**. The slope states, evaluated using r_d under increasing shaking magnitudes, are also discussed, followed by a 3D theoretical analysis.

Lastly, the findings presented in this part are listed in **Section 7.5**.

The entire sliding wedge's weight, W , the toe weight, W_2 , and the upper part's weight, W_1 , can be obtained using **Eq. (7-5)**, **Eq. (7-6)** and **Eq. (7-7)**, respectively. B represents the model breadth, ρ_t denotes the total unit weight, n is the scaling factor in centrifuge modeling to account for gravity, and g represents the gravitational acceleration in these calculations.

$$W = \frac{1}{2} \rho_t n g h^2 (\cot(\alpha) - \cot(\beta)) B \quad (7-5)$$

$$W_2 = \frac{1}{2} L_2 T B \rho_t n g \quad (7-6)$$

$$W_1 = W - W_2 \quad (7-7)$$

The unbalanced force from the upper part along the sliding surface, exerting on the virtual plane, F_P , is given in **Eq. (7-11)**. Here, F_{D1} represents the driving load of the upper section due to self-weight, while $S_{\alpha 1}$ is the resisting force provided by the normal force F_{N1} and cohesion c , assuming that the ground's shear strength is characterized by cohesion and friction angle ϕ , following the Mohr-Coulomb failure envelope.

$$F_{D1} = W_1 \sin(\alpha) + k_h W_1 \cos(\alpha) \quad (7-8)$$

$$F_{N1} = W_1 \cos(\alpha) - k_h W_1 \sin(\alpha) \quad (7-9)$$

$$S_{\alpha 1} = c L_1 B + F_{N1} \tan(\phi) \quad (7-10)$$

$$F_P = F_{D1} - S_{\alpha 1} \quad (7-11)$$

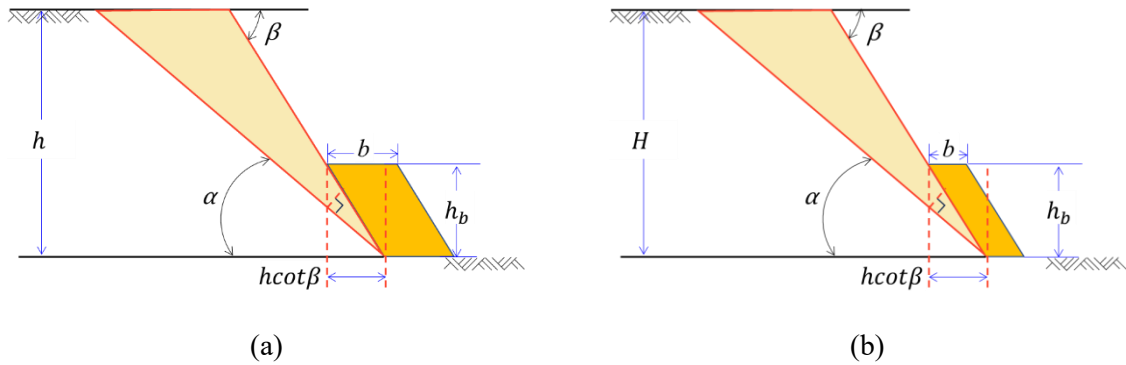


Fig. 7-2 Different first bench's width compared to the toe width: (a) larger than the toe width; (b) smaller than the toe width

The resisting force from the first bench is contributed by two parts, which was verified in **Section 5.4.3**. Here, Q represents the vertical overburden pressure produced by the part of the first bench on the uniform slope surface, and R is the horizontal resisting force caused by the remaining volume as a berm (see **Fig. 7-1**). When, $b \geq h \cot(\beta)$ as shown in **Fig. 7-2 (a)**, these two components can be obtained by **Eq. (7-12)** to **Eq. (7-14)**.

$$Q = \frac{1}{2} \rho_t n g h_b^2 \cot(\beta) B \quad (7-12)$$

$$W_R = \rho_t n g h_b B (b - \frac{1}{2} h_b \cot(\beta)) \quad (7-13)$$

$$R = c b B + W_R (\tan(\phi) - k_h) \quad (7-14)$$

When, $b < h \cot(\beta)$ as shown in **Fig. 7-2 (b)**, these two components can be obtained by **Eq. (7-15)** to **Eq. (7-17)**.

$$Q = \rho_t n g b (h_b - \frac{1}{2} b \tan(\beta)) B \quad (7-15)$$

$$W_R = \frac{1}{2} \rho_t n g b^2 \tan(\beta) B \quad (7-16)$$

$$R = c b B + W_R (\tan(\phi) - k_h) \quad (7-17)$$

Given the thrust force produced by the upper part, F_P , and the additional resisting forces applied by the first bench, Q and R , the total resisting force of the toe part, S_P , and the driving force, F_{D2} is obtained using **Eq. (7-18)** to **Eq. (7-20)**. Finally, the factor of safety (FS) is determined by **Eq. (7-21)**

$$N_P = (W_2 + Q) \cos(\alpha) + R \sin(\alpha) - k_h (W_2 + Q) \sin(\alpha) \quad (7-18)$$

$$S_P = c L_2 B + N_P \tan(\phi) + R \cos(\alpha) \quad (7-19)$$

$$F_{D2} = F_P + W_2 \sin(\alpha) + k_h W_2 \cos(\alpha) \quad (7-20)$$

$$FS = \frac{S_P}{F_{D2}} \quad (7-21)$$

7.1.2 Pseudo-static analysis for benched slope under 3D conditions

Following **Eq. (6-11)**, the FS for modes ① and ④ can be determined, while the FS for mode ③ can be obtained using an equivalent slope, as explained in **Section 5.1.1**. Referring to Pipatpongsa et al. (2023), the method for calculating FS for mode ② is detailed below.

For a soil wedge lying on the sliding surface, the average normal pressure, p_N can be approximated by **Eq. (7-22)** and the average of the lateral earth pressure can be approximated based on the developed passive earth pressure as **Eq. (7-23)**.

$$p_N = \rho_t n g T \cos(\alpha) \quad (7-22)$$

$$p_D = \frac{1}{2} p_N K + 2 r_d c \sqrt{K} \quad (7-23)$$

The triangular toe part below the virtual plane is taken out with two lateral surfaces, as illustrated in

Fig. 7-3. On lateral surfaces, differential lateral shear resistance on a differential slice having a thickness, dt , is expressed by **Eq. (7-24)**, integrating which leads to the expression of lateral resistance on the slope toe as **Eq. (7-25)**. Consequently, the total resisting force of the toe part, S_P is increased to **Eq. (7-26)**. Then, the FS can be calculated by **Eq. (7-21)**.

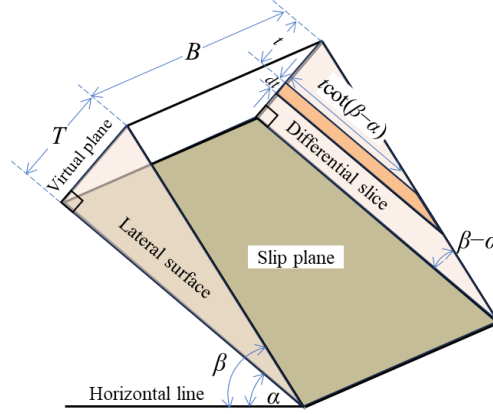


Fig. 7-3 Failure mechanism for the laterally confined toe part

$$dS_h = (p_D \tan(\phi_i) + c_i) t \cot(\beta - \alpha) dt \quad (7-24)$$

$$\begin{aligned} S_h &= \int_0^T (p_D \tan(\phi_i) + c_i) t \cot(\beta - \alpha) dt \\ &= \left(\left(\frac{K \rho_t n g T}{3} \cos(\alpha) + 2 r_d c \sqrt{K} \right) \tan(\phi_i) + c_i \right) \frac{T^2}{2} \cot(\beta - \alpha) \end{aligned} \quad (7-25)$$

$$S_P = c L_2 B + N_P \tan(\phi) + R \cos(\alpha) + 2 S_h \quad (7-26)$$

7.2 Experimental scheme

7.2.1 Test program

Centrifugal model tests were carried out to investigate the dynamic stability of benched slopes. The centrifuge facility utilized for these tests is located at the Geotechnical Centrifuge Center, Disaster Prevention Research Institute (DPRI), Kyoto University, featuring an effective radius of 2.5 m, maximum payload of 24 G×ton and a maximum centrifugal acceleration of 200G (Tao & Pipatpongsa, 2022). The attached shaking table to the centrifuge has a maximum capacity of 142.5 kg at 50G for dynamic excitation.

Two groups of centrifugal models were prepared, involving seven model slopes with an external first bench and two model slopes with equivalent excavation. For each case, two model slopes were tested simultaneously under symmetric seismic excitation to improve efficiency and to facilitate observation of the effects of varying slope angles and bench configurations. **Fig. 7-4** and **Fig. 7-5** show the model slopes prior to loading, with each model 9 m in a total height and 7.5 m in a slope height on the prototype scale at 50G. All cases used damp Hiroshima sand under with $D_c=90\%$ and $w=8\%$. Notably, lateral support for the first bench was removed in Case 5 of the first group. In Case 7, dry udon noodles were

embedded in the model to visualize the sliding surface. A narrow soil chamber was used for all cases in this chapter (see **Section 4.2.1** for details).

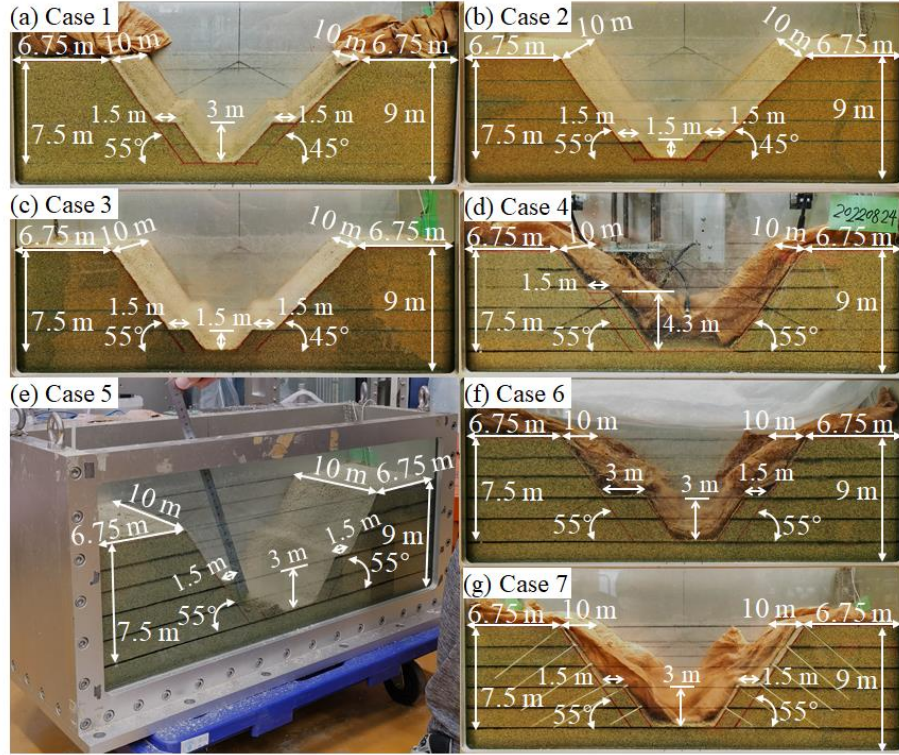


Fig. 7-4 Established model slopes before loading in the first group (prototype scale at 50G)

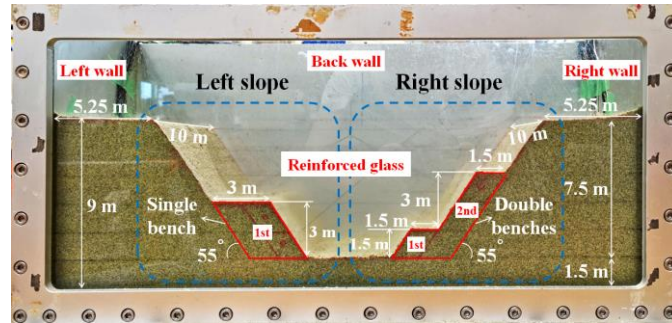


Fig. 7-5 Established model slopes before loading in the second group (prototype scale at 50G)

Table 7-1 and **Table 7-2** list the detailed model configurations for the seven cases in the first group, including the case number, experimental date, type of soil chamber used, degree of compaction (D_c), slope angle (β), height (B_H) and the width (B_W) of the first bench, initial water content (w), centrifugal loading, as well as the sequence of amplitude magnifications (explained in **Section 6.2.3**) for each test. A centrifugal loading of 50G was used in these tests. **Table 7-3** and

Table 7-4 present the configurations for the second group, with additional details about the second bench. **Fig. 7-6** illustrates the model slopes under centrifugal loading for both groups.

Table 7-1 Model configurations for benched slopes in the first group subject to ground motions (50G)

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)		1 st bench height, B_H (m)		1 st bench width, B_W (m)		Initial water content, w (%)	Loading (G)
				Left	Right	Left	Right	Left	Right		
1	220414	Narrow	90	55	45	3	3	1.5	1.5	8	50
2	220415			55	45	1.5	1.5	1.5	1.5		
3	220425			55	45	1.5	1.5	1.5	1.5		
4	220824			55	55	4.3	—	1.5	—		
5	221003			55	55	3	3	1.5	1.5		
6	221005			55	55	3	3	3	1.5		
7	221019			55	55	3	3	1.5	1.5		

Table 7-2 Amplitude magnification in the first group

No.	Date	Shaking magnification
1	220414	1.9→2.5→2.7→2.7→3.0→3.5
2	220415	1.9→2.5→2.7→2.7→3.0
3	220425	1.9→2.5→2.7→2.7→3.0
4	220824	1.9→2.5→2.7→2.7
5	221003	1.9→2.5→2.7
6	221005	1.9→2.5→2.7→2.7→3.0→3.5
7	221019	1.9→2.5→2.7

Table 7-3 Model configurations for benched slopes in the second group subject to ground motions

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)		1 st Bench height, B_{1H} (m)		1 st Bench width, B_{1W} (m)		2 nd Bench height, B_{2H} (m)		2 nd Bench width, B_{2W} (m)		Initial water content, w (%)	Loading (G)
				Left	Right	Left	Right	Left	Right	Left	Right	Left	Right		
1	230405	Narrow	90	55	3	1.5	3	1.5	—	3	—	1.5	8	50	
2	230721														

Table 7-4 Amplitude magnification in the second group

No.	Date	Shaking magnification
1	230405	1.9→2.5→2.7→3.0
2	230721	2.2→2.3→2.5→2.7→3.0→3.5

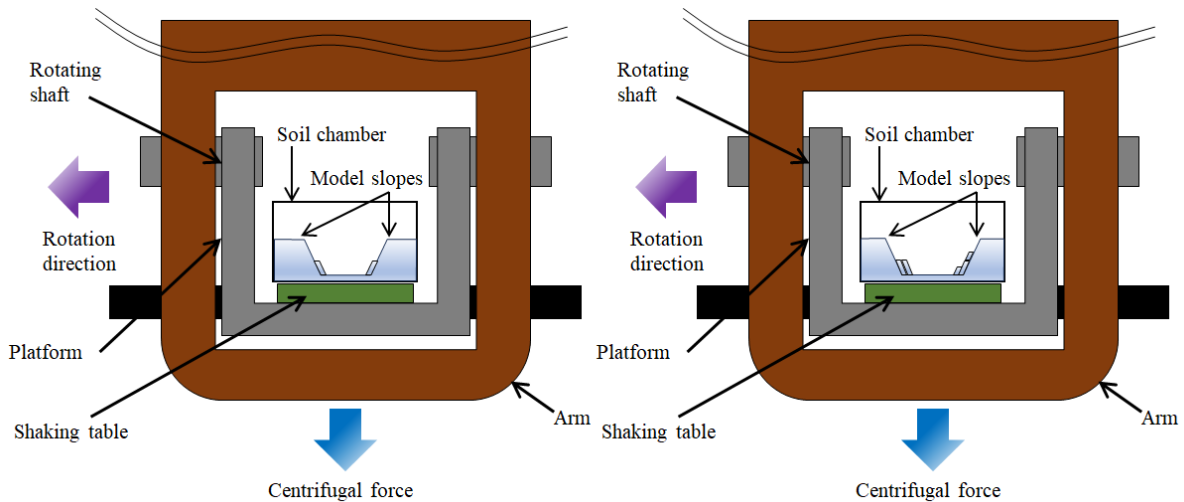


Fig. 7-6 Diagram of model slopes during centrifugal loading: (a) the first group; (b) the second group

7.2.2 Instrumentation

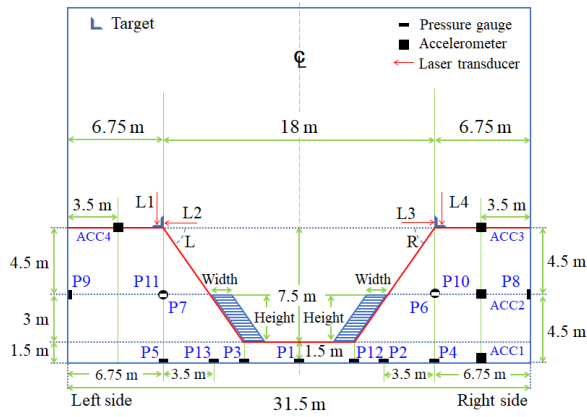
The specifications for the earth pressure gauges and laser sensors used in this chapter are consistent with those detailed in **Section 4.2.2**. Additionally, a maximum of six A6H-50G accelerometers were installed to record the response of model slopes during seismic excitation.

Up to thirteen earth pressure transducers were placed within the slopes to detect variation of internal earth pressure throughout each shaking event. Additionally, up to six laser sensors were strategically placed to record the displacement of the slope at specific locations.

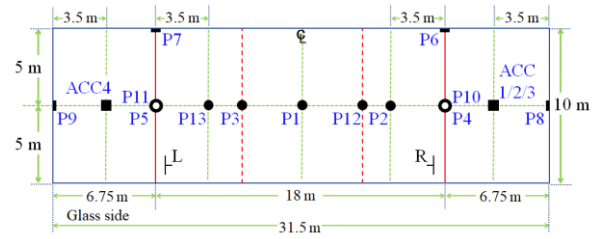
Sensor arrangement differed slightly across each group, with most sensors positioned at the bottom and middle heights of the model (see **Fig. 7-7**, **Fig. 7-8** and **Fig. 7-9**). Four earth pressure sensors (P6, P7, P8 and P9) were placed along the slope boundary to monitor horizontal earth pressures, while other nine earth pressure sensors (P1, P2, P3, P4, P5, P10, P11, P12 and P13) were used to monitor the vertical earth pressures.

The model slope displacements were measured using laser sensors. In the first six cases of the first group, four laser sensors measured the slope crests' vertical and horizontal displacements on both sides. In Case 7 of the first group, two additional sensors (LD3 and LD4) were installed to monitor the slope surfaces' horizontal displacement at the middle height, aligning the sensor configuration with that of the second group. Targets introduced in **Section 4.2.2** were also placed to enhance measurement accuracy.

high-speed cameras were used in each case to record slope deformation, providing visual evidence to complement the sensor measurements and allowing for a detailed analysis of the sliding progression and model behavior during tests.

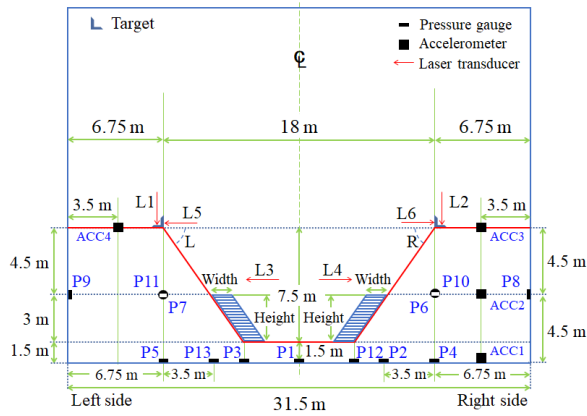


(a) Front view

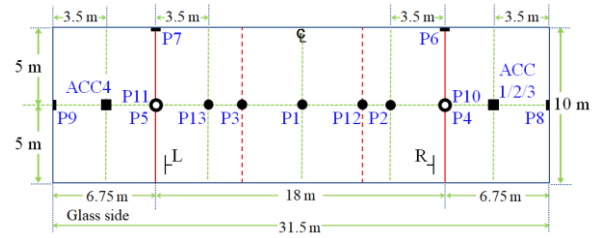


(b) Top view

Fig. 7-7 Configuration of sensors in Case 1 to Case 6 of the first group

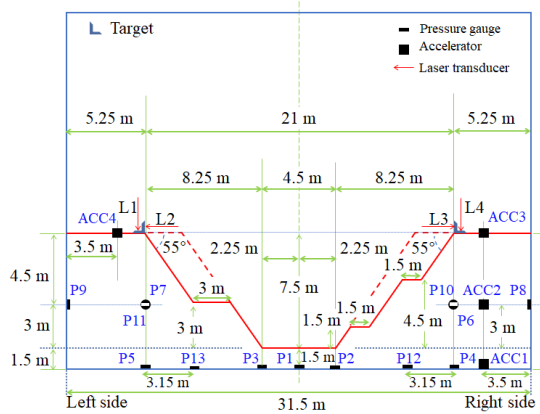


(a) Front view

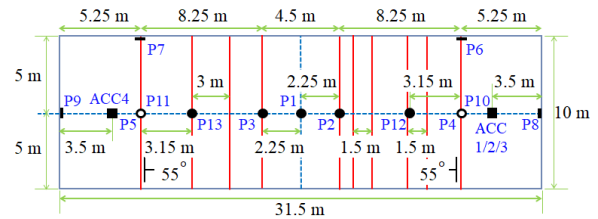


(b) Top view

Fig. 7-8 Configuration of sensors in Case 7 of the first group



(a) Front view



(b) Top view

Fig. 7-9 Configuration of sensors for two cases of the second group

The configuration of sensors for all cases are summarized in **Table 7-5** and **Table 7-6**.

Table 7-5 List of sensors of the first group

No.	Date	Earth pressure sensor													Accelerometer						Laser sensor					
		P	P	P	P	P	P	P	P	P	P	P	P	P	AC	AC	AC	AC	AC	AC	LD	LD	LD	LD	LD	LD
		1	2	3	4	5	6	7	8	9	10	11	12	13	C1	C2	C3	C4	C5	C6	1	2	3	4	5	6
1	220414	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	—	○	○	○	○	—	—
2	220415	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	○	○	○	○	○	—	—
3	220425	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	○	○	○	○	○	—	—
4	220824	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	—	○	○	○	○	—	—
5	221003	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—	○	○	○	○	—	—
6	221005	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—	○	○	○	○	—	—
7	221019	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned

- The sensor was not placed

Table 7-6 List of sensors for all cases in the second group

No.	Date	Earth pressure sensor												Accelerometer						Laser sensor						
		P	P	P	P	P	P	P	P	P	P	P	P	AC	AC	AC	AC	AC	AC	LD	LD	LD	LD	LD	LD	
		1	2	3	4	5	6	7	8	9	10	11	12	13	C1	C2	C3	C4	C5	C6	1	2	3	4	5	6
1	230405	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	—	○	○	○	○	—	—
2	230721	○	○	○	○	○	○	○	○	○	○	○	—	—	○	○	○	○	○	○	○	○	○	○	—	—

Note: ○ The sensor was placed and functioned △ The sensor was placed but malfunctioned

- The sensor was not placed

7.2.3 Test procedure

The experimental procedure followed a consistent process. Initially, dry Hiroshima sand was blended with water to achieve a desired water content of 8%. Prepared sand was then divided into six equal portions and compacted in the soil chamber in six layers, each layer having a height of 3 cm (model scale). A wooden hammer was used for compaction, and the weight required for each layer was determined from the degree of compaction ($D_c=90\%$) and the layer's volume. Sensors were installed as the soil was compacted to specific heights. After forming a horizontal foundation with a height of 18 cm (model scale), unnecessary soil was excavated from the center, and two slope surfaces were formed using a spatula. The preparation procedure is shown in **Fig. 7-10**. In Group 1, the benched slope was constructed by adding the bench part to the original slope, while in Group 2, the benched slope was shaped by excavating from the original slope.

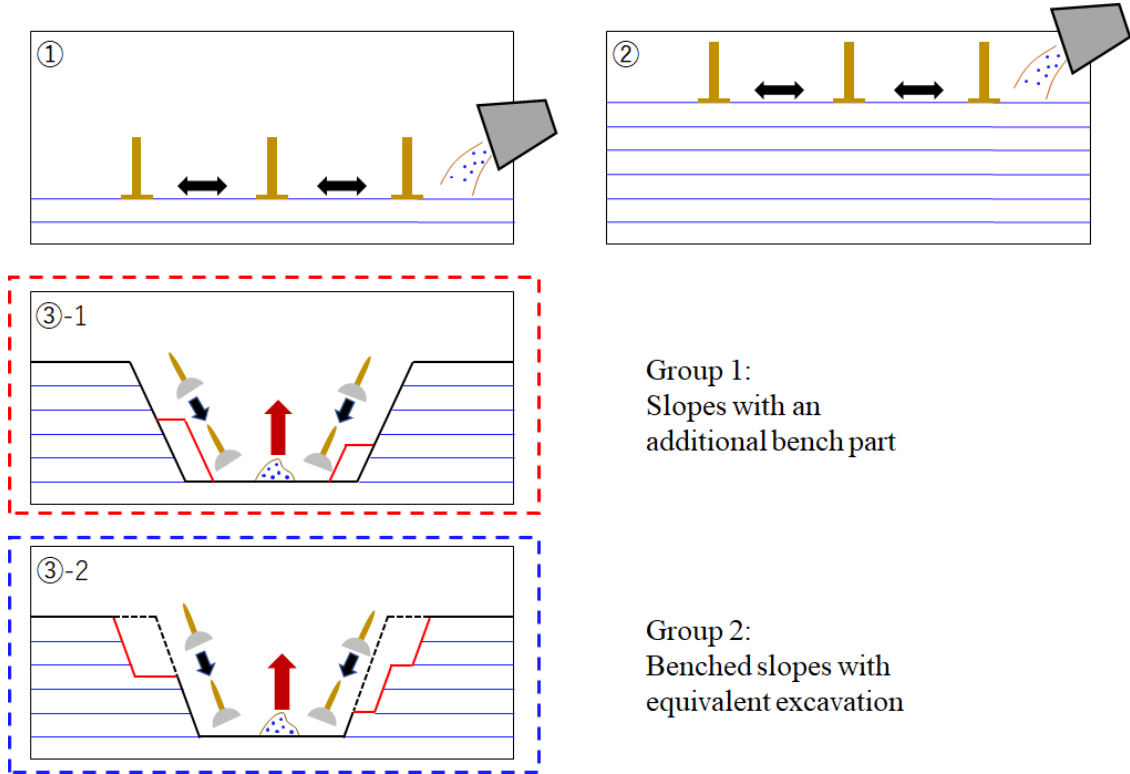


Fig. 7-10 Flow chart of the preparation procedure

After installation onto the centrifuge platform, the model underwent a gradual increase in centrifugal loading, starting from 1G and progressing steadily to 50G. At this stage, the centrifugal acceleration was maintained until the sensor values stabilized. Following this stabilization, seismic excitations, controlled by displacement, were applied sequentially using an inflight shaking table. Each excitation used a datum ramped sine wave, as detailed in **Section 6.2.3**, with amplitudes gradually magnified for each wave input. The specific magnifications for every case are provided in **Table 7-2** and **Table 7-4**.

7.3 Experimental Results

The high-speed camera captured detailed images of the slope sliding process, with all sliding events occurring within one second. These events are presented in sequential order for each case. The maximum absolute acceleration value, recorded by the accelerometer at the model slope's base (ACC1), serves as an indicator of each input shaking's intensity. Additionally, sensor-measured data on displacement and earth pressure are provided to further illustrate the slope's response during the tests.

7.3.1 Laterally confined benched slopes with an additional bench part

In each case, β_L and β_R denote left and right slopes' inclinations, respectively. B_{HL} and B_{WL} represent the left first bench's height and width. B_{HR} and B_{WR} are the right first bench's height and width.

7.3.1.1 Case 1 ($\beta_L=55^\circ$, $\beta_R=45^\circ$, $B_{HL}=3$ m, $B_{WL}=1.5$ cm, $B_{HR}=3$ m, $B_{WR}=1.5$ cm)

Sliding process

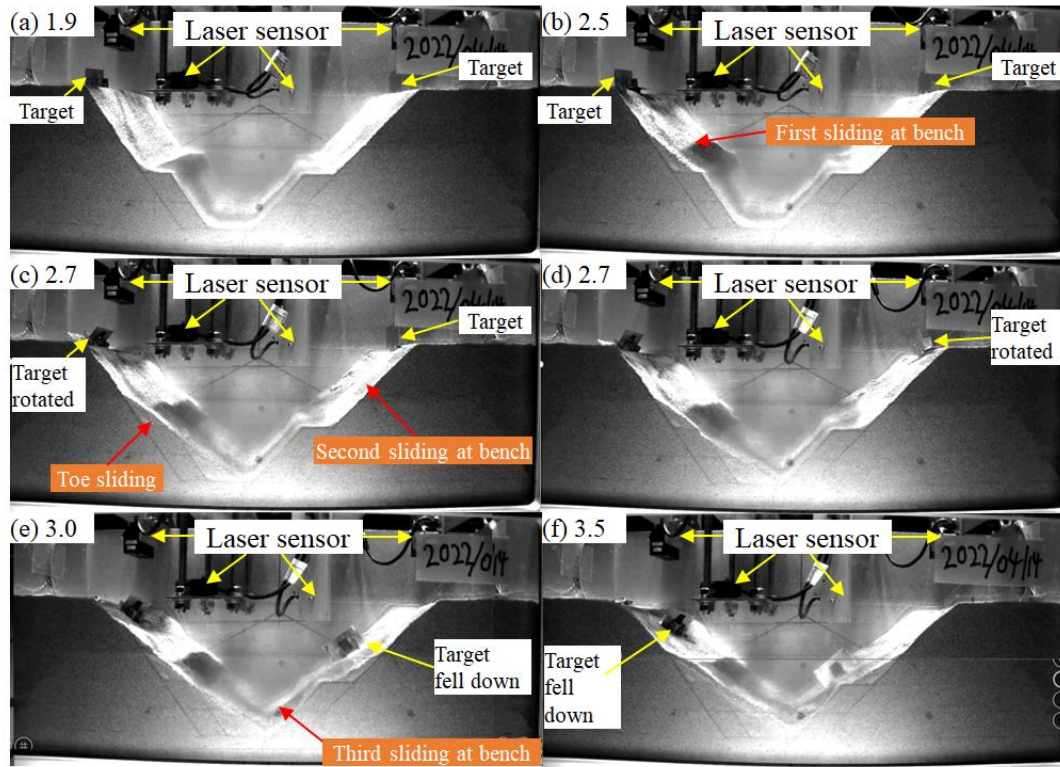


Fig. 7-11 Reaction of the model slopes to each shaking in Case 1 of the first group: (a) stable state after the first shaking; (b) first sliding at bench on the left slope; (c) toe sliding on the left slope and second sliding at bench on the right slope; (d) state unchanged (e) third sliding at bench on the right slope (f) last shaking

The types of sliding are defined as follows:

Toe Sliding: The sliding surface connects the slope crest and the toe, across the entire breadth.

Local Toe Sliding: The sliding surface connects the slope crest and the toe but only across partial slope breadth.

Bench sliding: The sliding area is restricted to the bench level only.

In Case 1 of the first group, six seismic excitations with increasing amplitudes were applied. The acceleration data from ACC1 was converted into the horizontal seismic coefficient, k_h , in prototype scale, by dividing by 50g. The corresponding k_h values when sliding happened are shown in **Fig. 7-12**. Due to an operational error, data from the fourth shaking was not recorded.

The model slopes remained stable after the first shaking, with a magnification of 1.9 ($k_h=0.334$) (see **Fig. 7-11 (a)**). The second shaking, with a magnification of 2.5 ($k_h=0.447$) (see **Fig. 7-11 (b)**), induced sliding at the left second bench. During the third shaking, with a magnification of 2.7 ($k_h=0.505$), toe sliding occurred on the left slope, and the right second bench also failed. The fourth

shaking, with the same magnification as the third, did not alter the model state.

The fifth shaking, with a magnification of 3.0 ($k_h=0.588$), caused failure of the first bench on the right slope. However, even after the final shaking at a magnification of 3.5 ($k_h=0.680$), no continuous toe sliding from the right slope crest to the toe was observed.

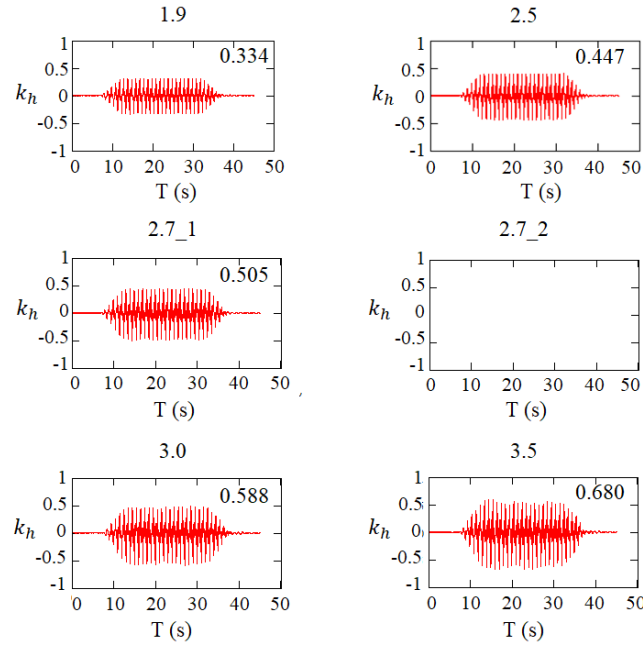


Fig. 7-12 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 1 of the first group

Displacement

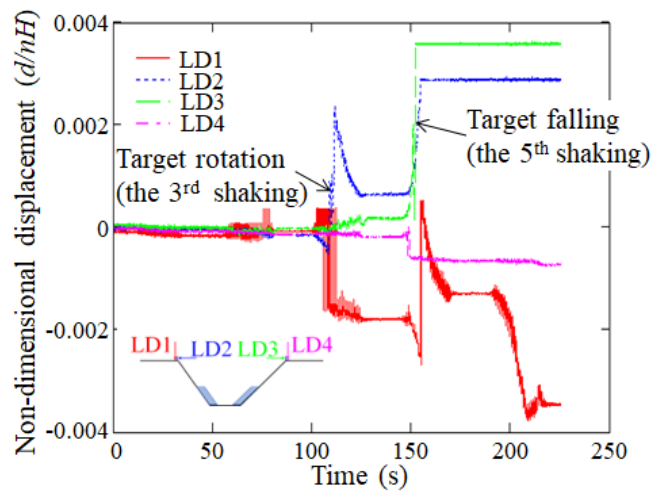


Fig. 7-13 Vertical and horizontal displacement of slope crests with each shaking in Case 1 of the first group

The relationship between non-dimensional displacement, d/nH , and each shaking is presented in **Fig. 7-13**. Here, d denotes the actual displacement measured by the laser sensor, $n = 50$ is the scaling factor in centrifuge modeling to account for gravity, and H represents the total height of the model, which is 18 cm (model scale). Positive values indicate that the target, representing the displacement of the slope crest, towards the laser sensor, while a negative value indicates movement away from it.

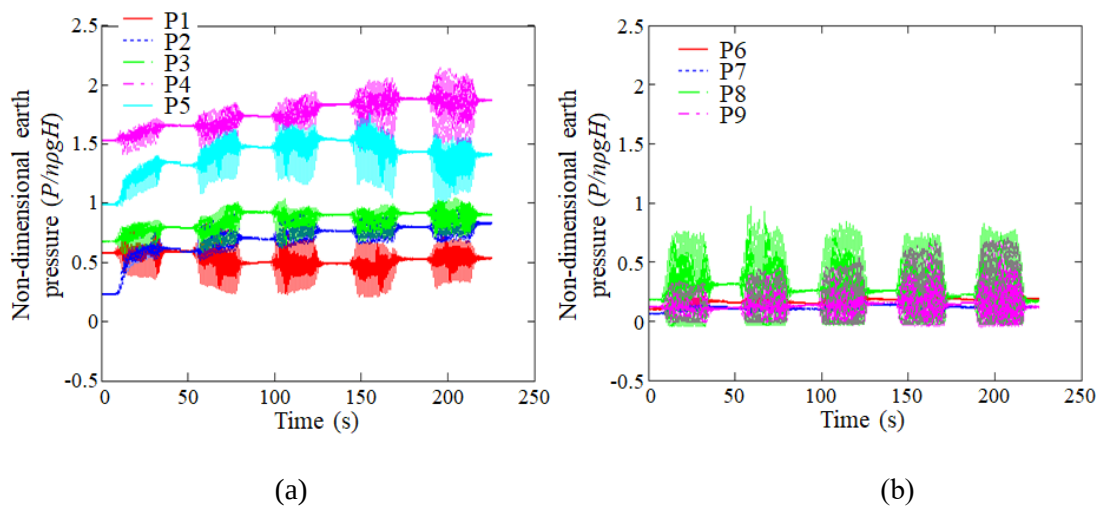
Each shaking led to further settling of the crest and forward deformation. During the third shaking, the rotation of the target on the left crest resulted in abrupt changes in LD1 and LD2. Following the fifth shaking, the targets on both slopes were lost, causing LD2 and LD3 to go out of range.

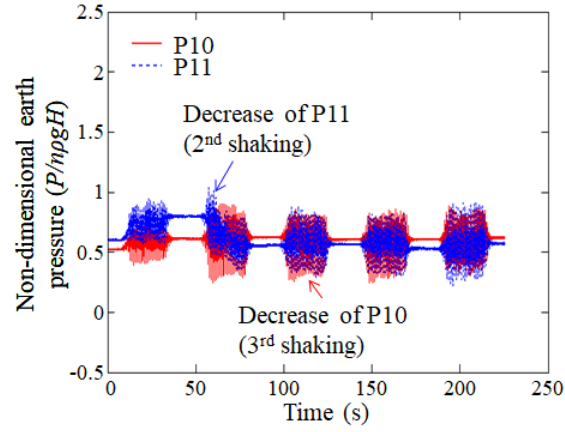
Earth pressure

The variation in non-dimensional earth pressure, $P/n\rho gH$, is plotted with respect to each shaking in **Fig. 7-14** for Case 1 of the first group. Here, P is the earth pressure measured by the sensors, and g represents gravitational acceleration. Earth pressures fluctuate with each shaking, and its value can change before and after shaking.

The post-shaking values for sensors P1 to P5, positioned at the bottom of the model, show a gradual increase after each shaking, while the other pressures exhibit less variation. Sensors P6 to P9 display minimal change between their pre-shaking and post-shaking values throughout the process. P10 and P11 demonstrate a unique pattern: their post-shaking values are initially higher than their pre-shaking values, but after certain shakings, the post-shaking values decrease below the pre-shaking levels. For P10, this decrease appears after the third shaking, while for P11, it occurs after the second shaking.

As noted, the sliding on the right first bench occurred in the second shaking and during the third shaking on the left slope. This reduction in post-shaking earth pressure corresponds with the occurrence of sliding, indicating a release of pressure following these events.





(c)

Fig. 7-14 Measured earth pressure with each shaking in Case 1 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 7-15 illustrates the measured accelerations at distinct locations of the model for each shaking. ACCT records the output wave from the shaking table, and the acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is apparent, with acceleration values increasing at higher elevations within the slope.

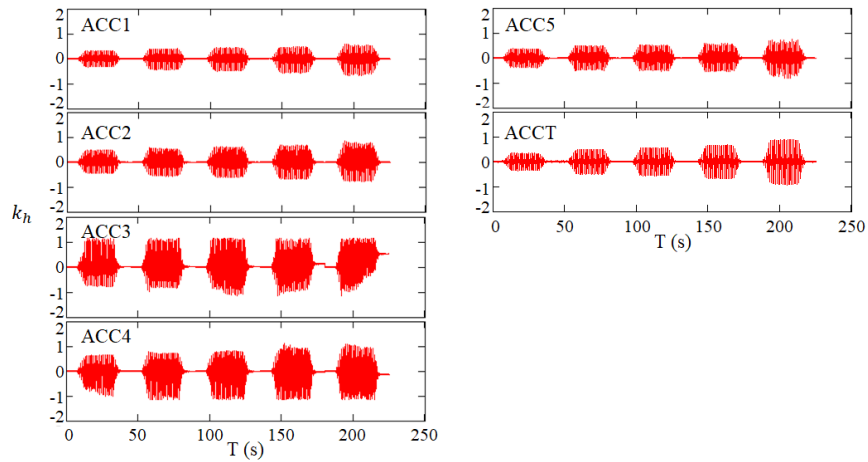


Fig. 7-15 Measured acceleration with each shaking in Case 1 of the first group

7.3.1.2 Case 2 ($\beta_L=55^\circ$, $\beta_R=45^\circ$, $B_{HL}=1.5$ m, $B_{WL}=1.5$ cm, $B_{HR}=1.5$ m, $B_{WR}=1.5$ cm)

Sliding process

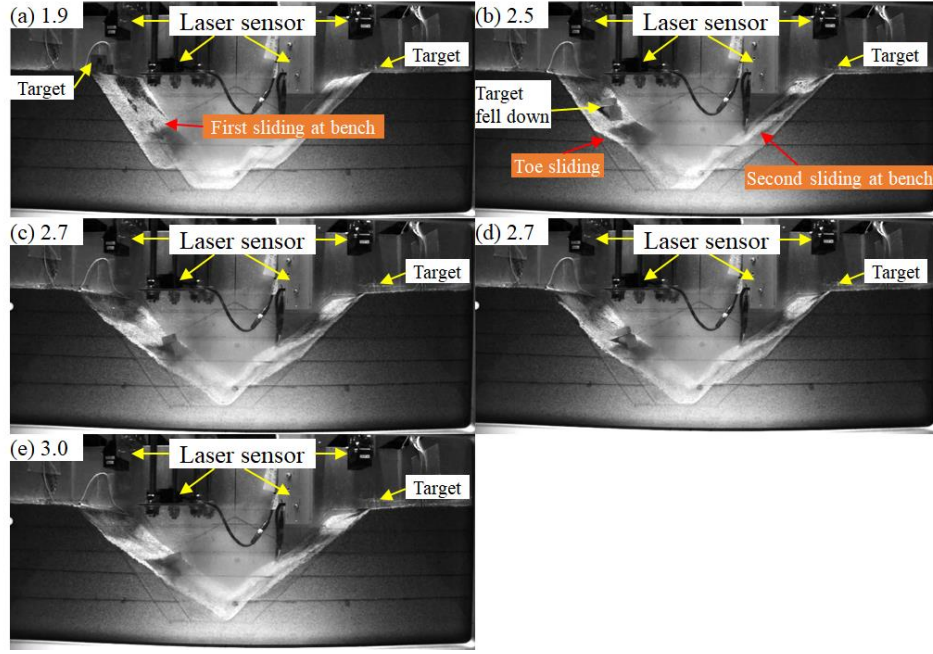


Fig. 7-16 Reaction of the model slopes to each shaking in Case 2 of the first group: (a) first sliding at bench on the left slope; (b) toe sliding on the left slope and second sliding at bench on the right slope; (c) state unchanged; (d) state unchanged; (e) state unchanged

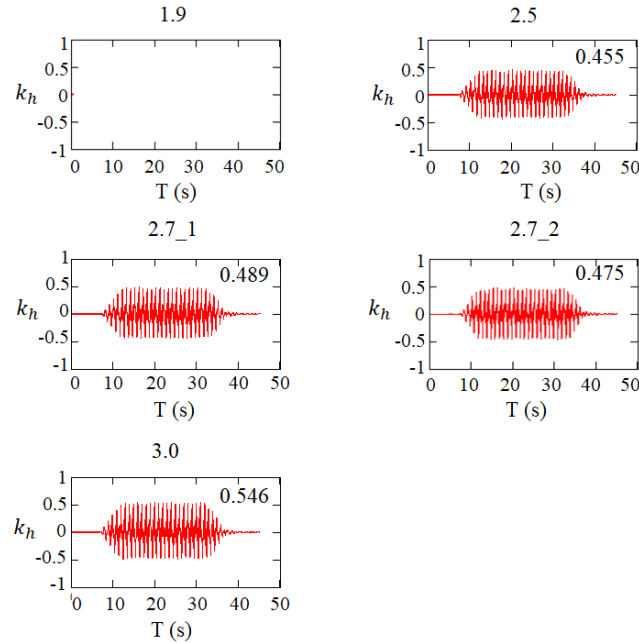


Fig. 7-17 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 2 of the first group

In Case 2 of the first group, the initial sliding occurred on the left second bench during the first shaking with a magnification of 1.9 (see **Fig. 7-16 (a)**), though data from this shaking was not recorded due to an error in operation. During the second shaking, with a magnification of 2.5 ($k_h=0.455$), toe

sliding occurred on the left slope, while the right second bench also failed (see **Fig. 7-16 (b)**). Subsequent shakings exacerbated the failure; however, the overall state of the model slopes remained unchanged.

Displacement

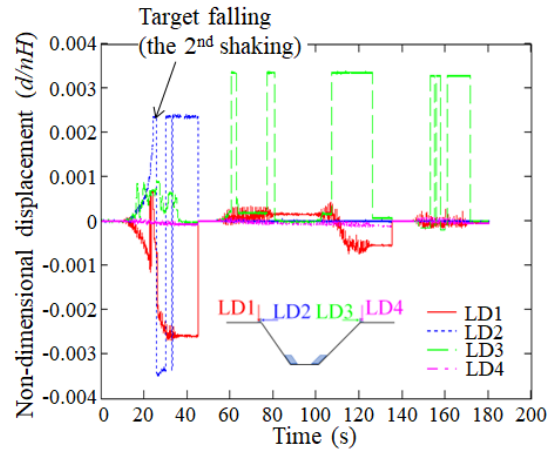
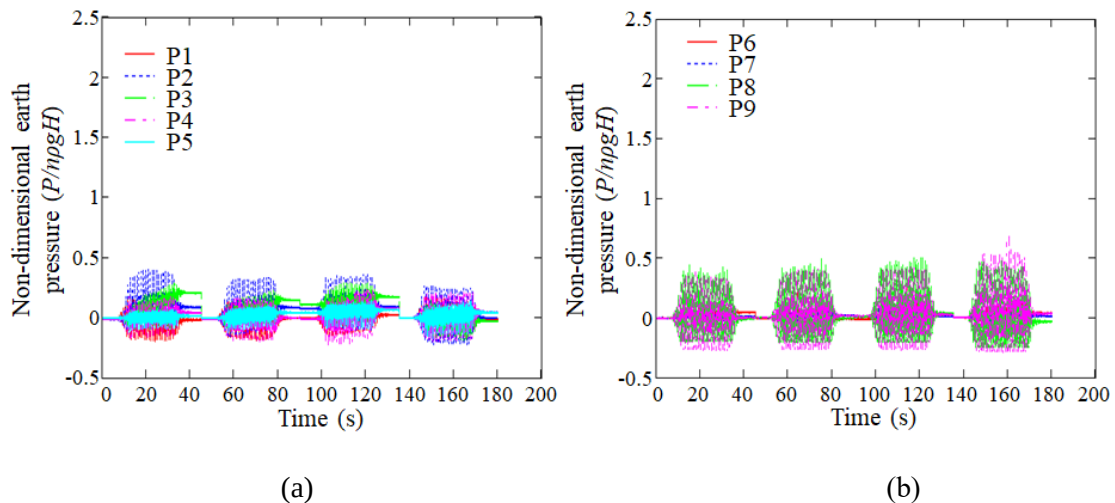
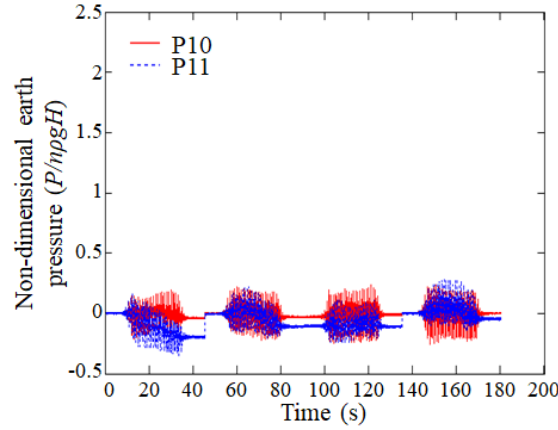


Fig. 7-18 Vertical and horizontal displacement of slope crests with each shaking in Case 2 of the first group

In **Fig. 7-18**, the symbols are consistent with those in **Fig. 7-13**, showing the variation of non-dimensional displacement with each shaking in Case 2 of the first group. For this case, all sensors were reinitialized before each shaking, so the values start from zero at the beginning and return to zero after each shaking. During the second shaking, LD2 went out of range, and LD1 immediately dropped due to the falling of the target on the left slope.

Earth pressure





(c)

Fig. 7-19 Measured earth pressure with each shaking in Case 2 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Fig. 7-19 illustrates the variation of non-dimensional displacement with respect to each shaking in Case 2 of the first group. In contrast to Case 1, all earth pressures in Case 2 fluctuate around zero, as they were reinitialized before and after each shaking (see **Fig. 7-19**).

Acceleration

Fig. 7-20 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave from the shaking table. The acceleration values have been scaled to the prototype scale k_h by dividing by 50g, with higher elevations showing increased acceleration,. ACC6 was positioned near ACC2 as a backup.

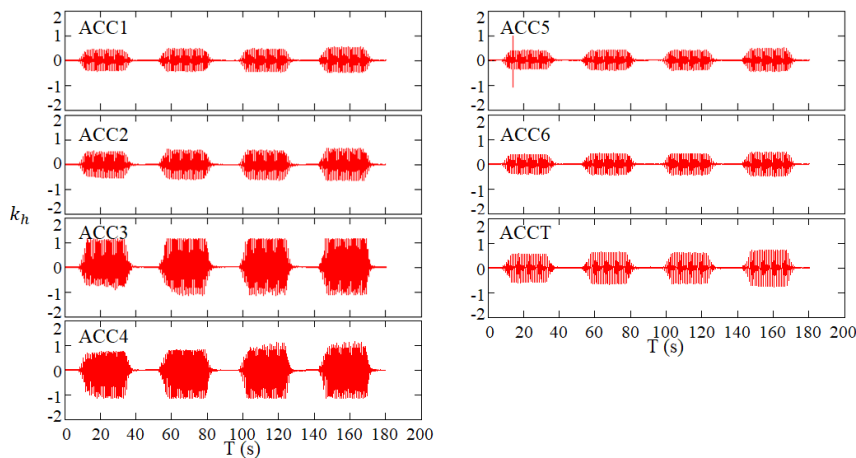


Fig. 7-20 Measured acceleration with each shaking in Case 2 of the first group

7.3.1.3 Case 3 ($\beta_L=55^\circ$, $\beta_R=45^\circ$, $B_{HL}=1.5$ m, $B_{WL}=1.5$ cm, $B_{HR}=1.5$ m, $B_{WR}=1.5$ cm)

Sliding process

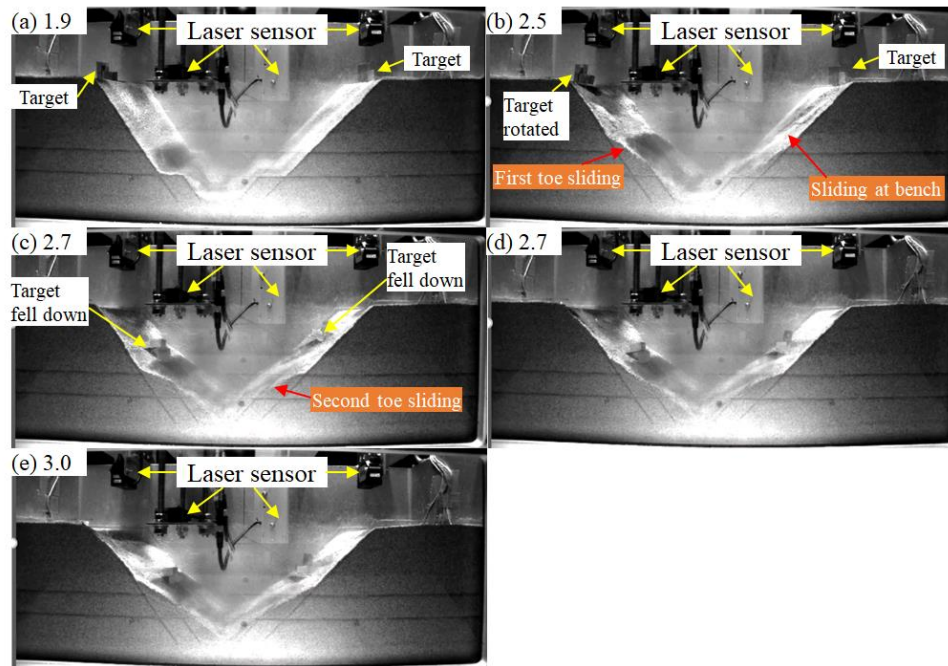


Fig. 7-21 Reaction of the model slopes to each shaking in Case 3 of the first group: (a) stable state after the first shaking; (b) first toe sliding on the left slope and sliding at bench on the right slope; (c) second toe sliding on the right slope; (d) state unchanged; (e) state unchanged

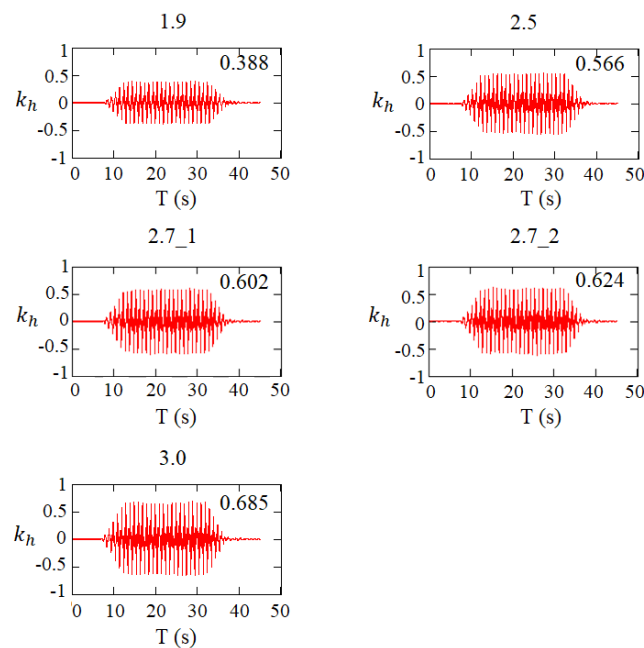


Fig. 7-22 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 3 of the first group

In Case 3 of the first group, both slopes remained stable following the first shaking, with a magnification of 1.9 ($k_h=0.388$) (see **Fig. 7-21 (a)**). Then, the second shaking, with a magnification of 2.5 ($k_h=0.566$) triggered toe sliding on the left slope and a bench sliding on the right second bench (see **Fig. 7-21 (b)**). During the third shaking, with a magnification of 2.7 ($k_h=0.602$), toe sliding occurred on the right slope (see **Fig. 7-21 (c)**). The following two shakings did not significantly alter the state of the model.

Displacement

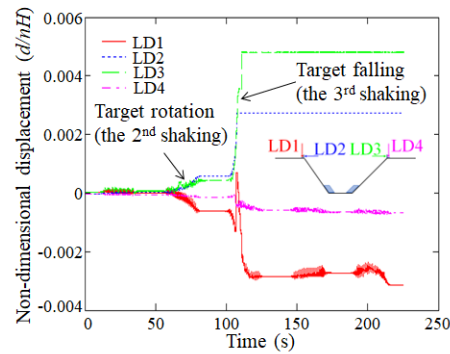
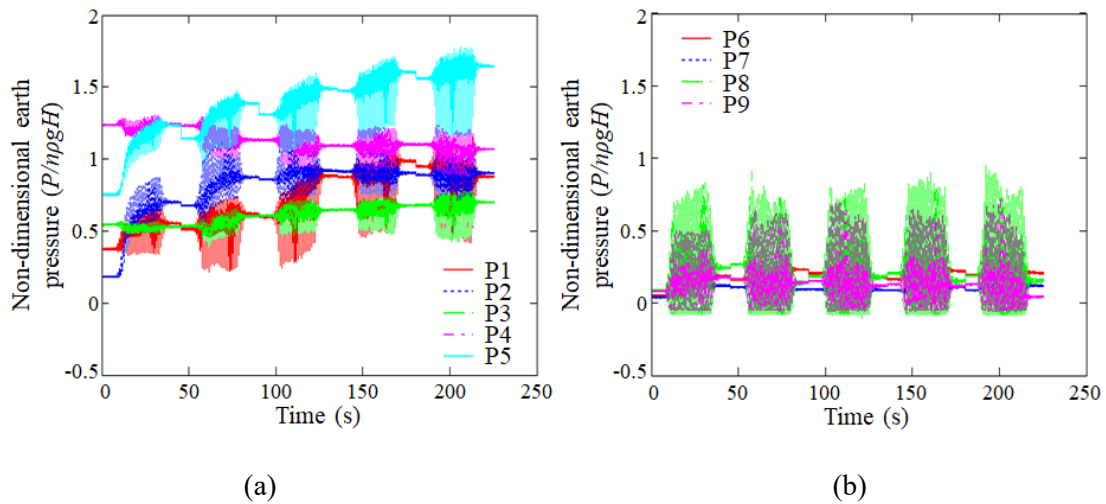
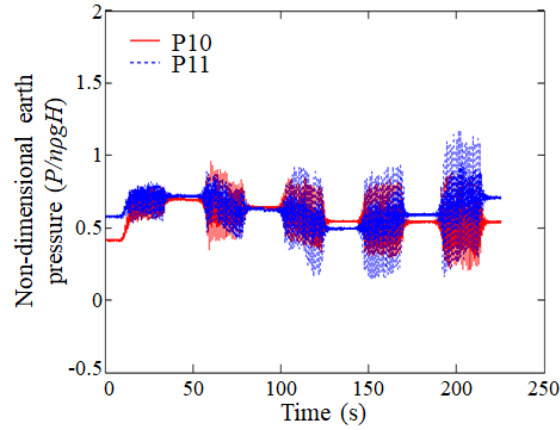


Fig. 7-23 Vertical and horizontal displacement of slope crests
with each shaking in Case 3 of the first group

In **Fig. 7-23**, the symbols can refer to those in **Fig. 7-13**, showing the variation of non-dimensional displacement with each shaking in Case 3 of the first group. During the second shaking, LD1 to LD4 exhibited changes due to target rotation. Subsequently, the third shaking caused both targets to fall, resulting in LD2 and LD3 going out of range, and immediate decreases in LD1 and LD4.

Earth pressure





(c)

Fig. 7-24 Measured earth pressure with each shaking in Case 3 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

In **Fig. 7-24**, the variation of non-dimensional earth pressure with each shaking in Case 3 of the first group is shown, with symbols representing the same elements as in **Fig. 7-14**.

Acceleration

In **Fig. 7-25**, the measured accelerations at various locations of the model slope during each shaking are illustrated. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations. ACC6 was placed near ACC2 as a backup.

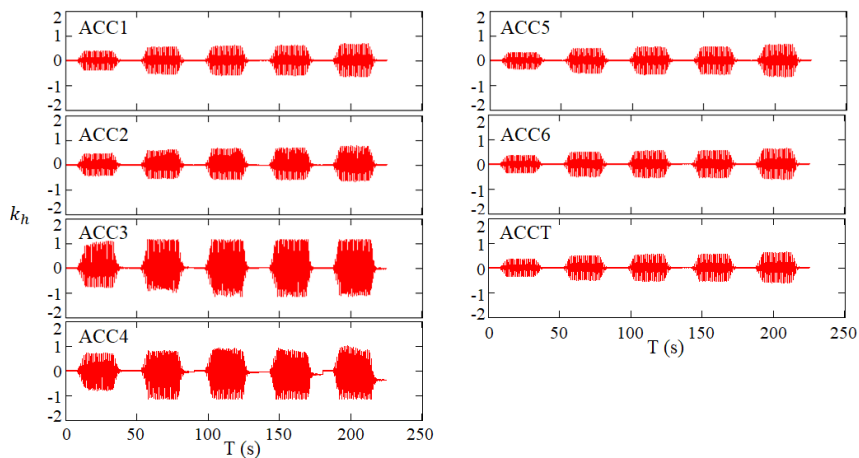


Fig. 7-25 Measured acceleration with each shaking in Case 3 of the first group

7.3.1.4 Case 4 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B_{HL}=4.3$ m, $B_{WL}=1.5$ cm)

Sliding process

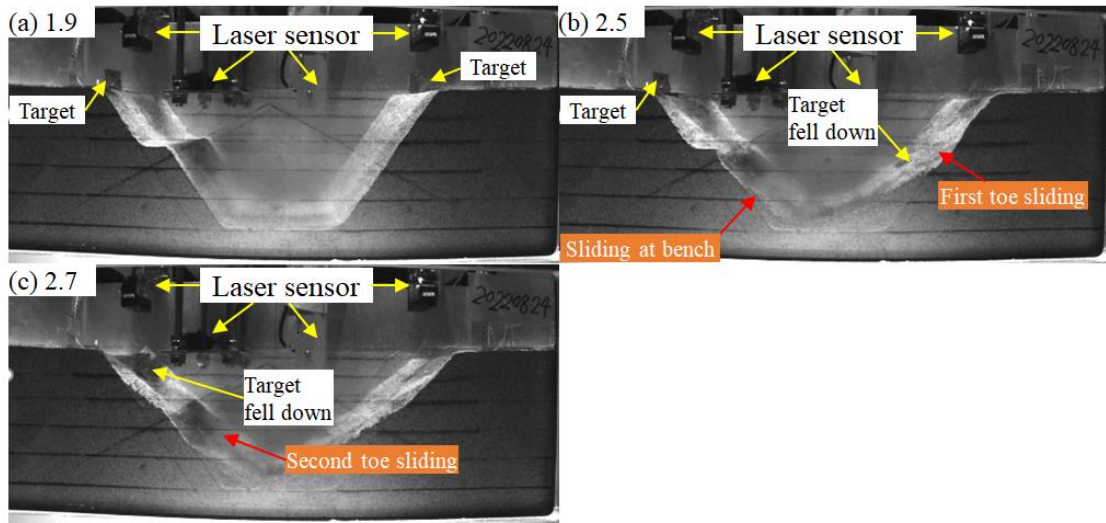


Fig. 7-26 Reaction of the model slopes to each shaking in Case 4 of the first group: (a) stable state after the first shaking; (b) sliding at bench on the left slope and first toe sliding on the right slope; (c) second toe sliding on the left slope

In Case 4 of the first group, the first bench height of the left slope was higher than other cases while the right-hand side was a uniform slope with the same slope angle. After the initial shaking with a magnification of 1.9 ($k_h=0.384$), both slopes remained stable (see **Fig. 7-26 (a)**). However, during the second shaking with a magnification of 2.5 ($k_h=0.500$), the left slope experienced failure at the first bench, and the right slope displayed a toe failure (see **Fig. 7-26 (b)**). Subsequently, the third shaking with a magnification of 2.7 ($k_h=0.541$) resulted in an toe failure on the left slope (see **Fig. 7-26 (c)**).

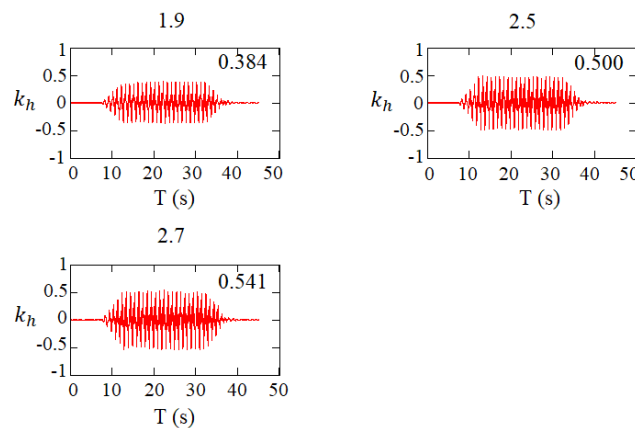


Fig. 7-27 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 4 of the first group

Displacement

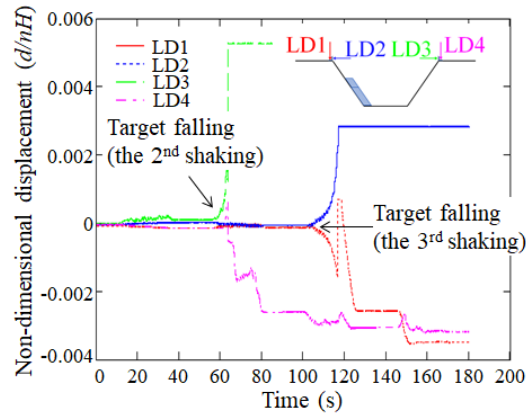
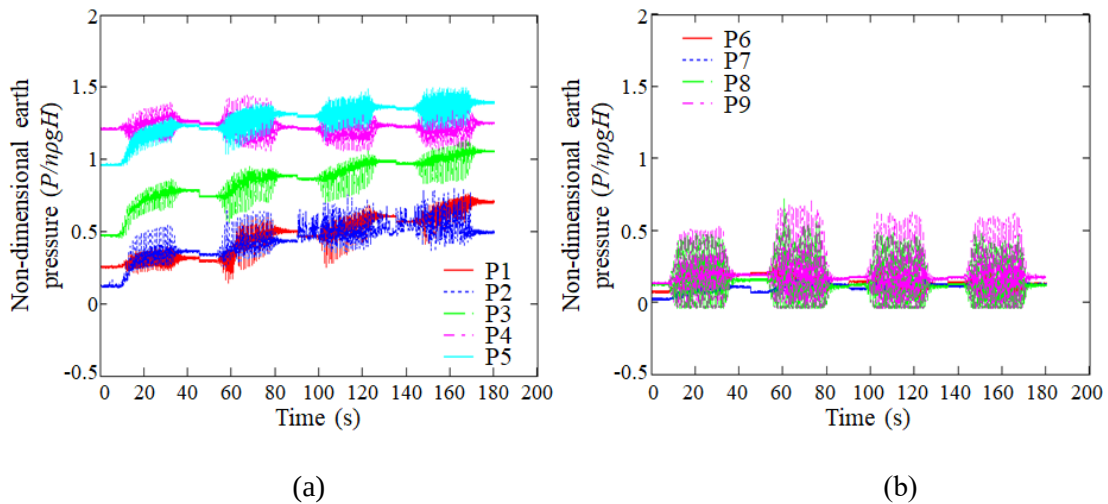
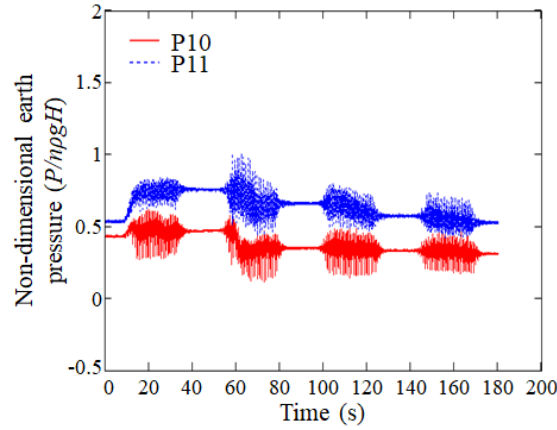


Fig. 7-28 Vertical and horizontal displacement of slope crests with each shaking in Case 4 of the first group

The symbols in **Fig. 7-28** have the same meanings as those in **Fig. 7-13**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 4 of the first group. During the toe sliding on the right slope, LD3 went out of range, and LD4 decreased due to the target falling. Similarly, during the third shaking, LD2 went out of range, and LD1 decreased.

Earth pressure





(c)

Fig. 7-29 Measured earth pressure with each shaking in Case 4 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Fig. 7-29 presents the change of non-dimensional earth pressure with each shaking in Case 4 of the first group, where the symbols can refer to those in **Fig. 7-14**.

Acceleration

Fig. 7-30 illustrates the measured accelerations at various locations of the model slope during each shaking. ACC2 records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

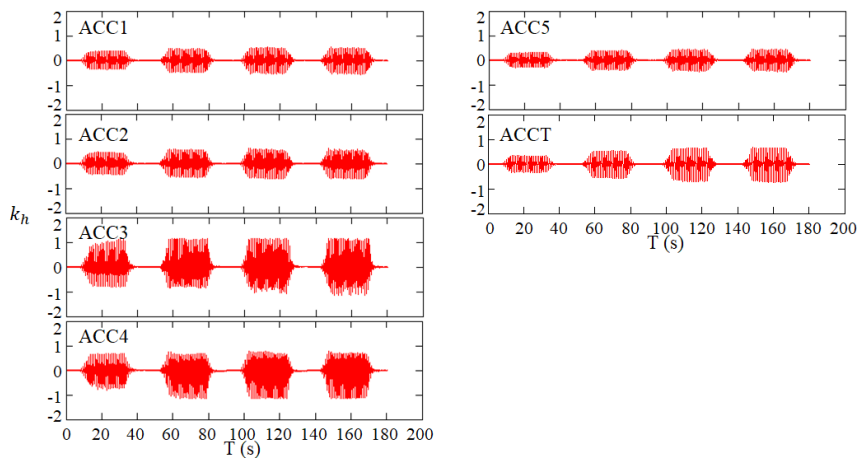


Fig. 7-30 Measured acceleration with each shaking in Case 4 of the first group

7.3.1.5 Case 5 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B_{HL}=3$ m, $B_{WL}=1.5$ cm, $B_{HR}=3$ m, $B_{WR}=1.5$ cm)

Sliding process

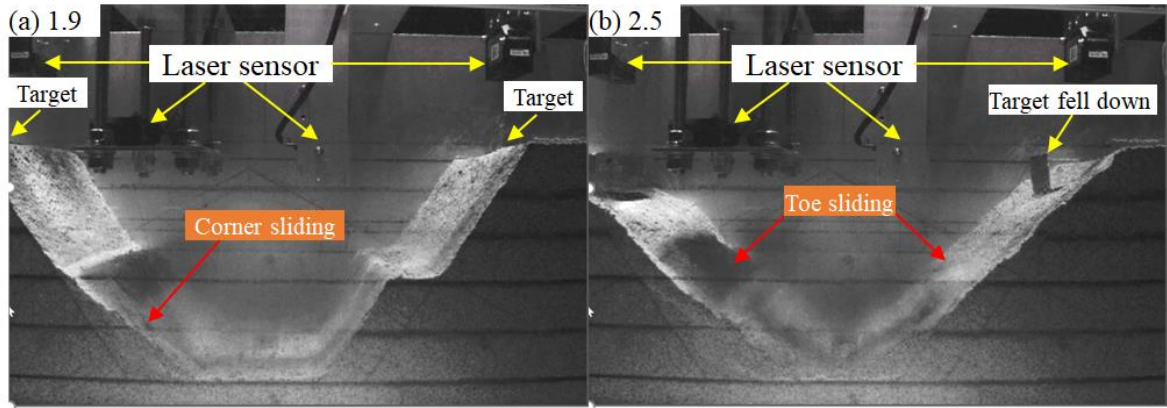


Fig. 7-31 Reaction of the model slopes to each shaking in Case 5 of the first group: (a) corner sliding on the left first bench; (b) toe sliding on both slopes

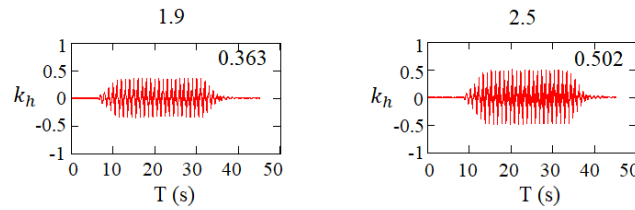


Fig. 7-32 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 5 of the first group

In Case 5 of the first group, the removal of lateral support from the first bench on the left slope led to initial corner sliding, which was then exacerbated after the first shaking at a magnification of 1.9 ($k_h=0.363$) (see **Fig. 7-31 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.502$), toe sliding occurred on both slopes (see **Fig. 7-31 (b)**). This case was specifically designed to assess the impact of lateral resistance provided by the first bench. The results suggest that lateral resistance from the first bench does not significantly impact the seismic stability of the benched slope and can be reasonably disregarded in stability evaluations.

Displacement

The symbols in **Fig. 7-33** have the same meanings as those in **Fig. 7-13**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 5 of the first group. Both targets fell during the second shaking, leading to a decrease in LD1 and LD4 values and causing LD2 and LD3 to go out of range.

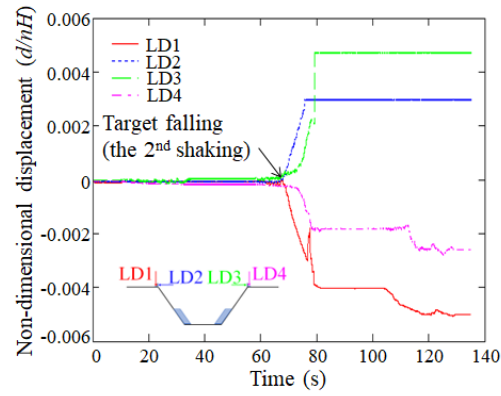


Fig. 7-33 Vertical and horizontal displacement of slope crests with each shaking in Case 5 of the first group

Earth pressure

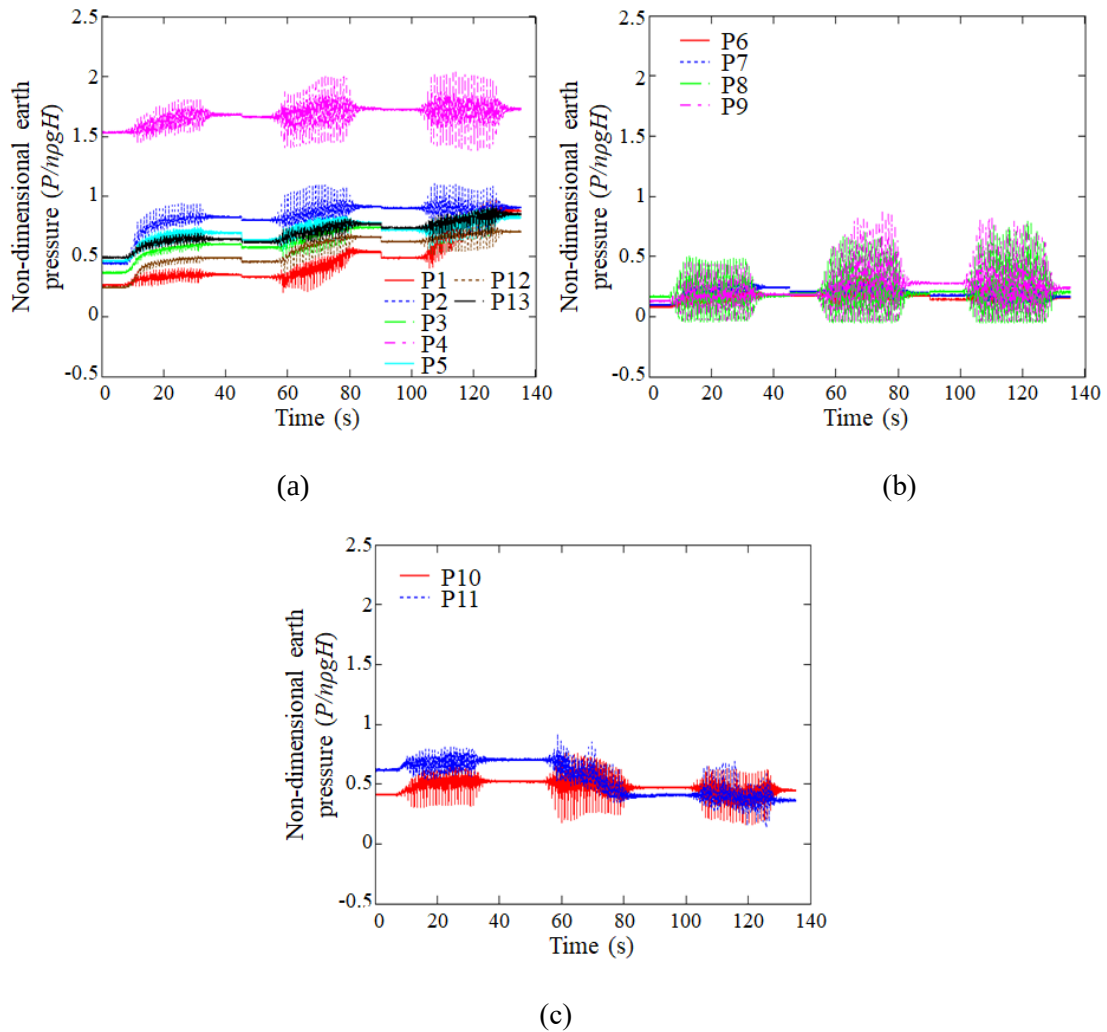


Fig. 7-34 Measured earth pressure with each shaking in Case 5 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

In **Fig. 7-34**, the variation of non-dimensional earth pressure with each shaking in Case 5 is shown, where the symbols can refer to those in **Fig. 7-14**. For this case, two additional earth pressure sensors (P12 and P13) were installed on the bottom of the model, providing more detailed monitoring of pressure changes during each shaking.

Acceleration

Fig. 7-35 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave from shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

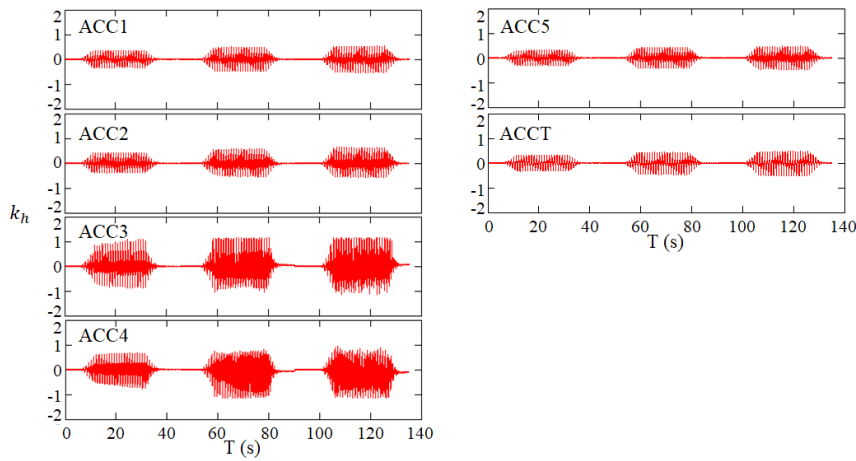


Fig. 7-35 Measured acceleration with each shaking in Case 5 of the first group

7.3.1.6 Case 6 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B_{HL}=3$ m, $B_{WL}=3$ cm, $B_{HR}=3$ m, $B_{WR}=1.5$ cm)

Sliding process

In Case 6 of the first group, the left first bench was twice the width of the right bench. Following the first shaking with a magnification of 1.9 ($k_h=0.376$), both slopes remained stable (see **Fig. 7-36 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.525$), a bench failure occurred on the left second bench, while the right slope experienced toe sliding (see **Fig. 7-36 (b)**). The third shaking with a magnification of 2.7 ($k_h=0.597$) caused failure of the first bench on the left slope (see **Fig. 7-36 (c)**). Since both slopes failed after the third shaking, the following three shakings did not affect the stability significantly and are not displayed in the figure.

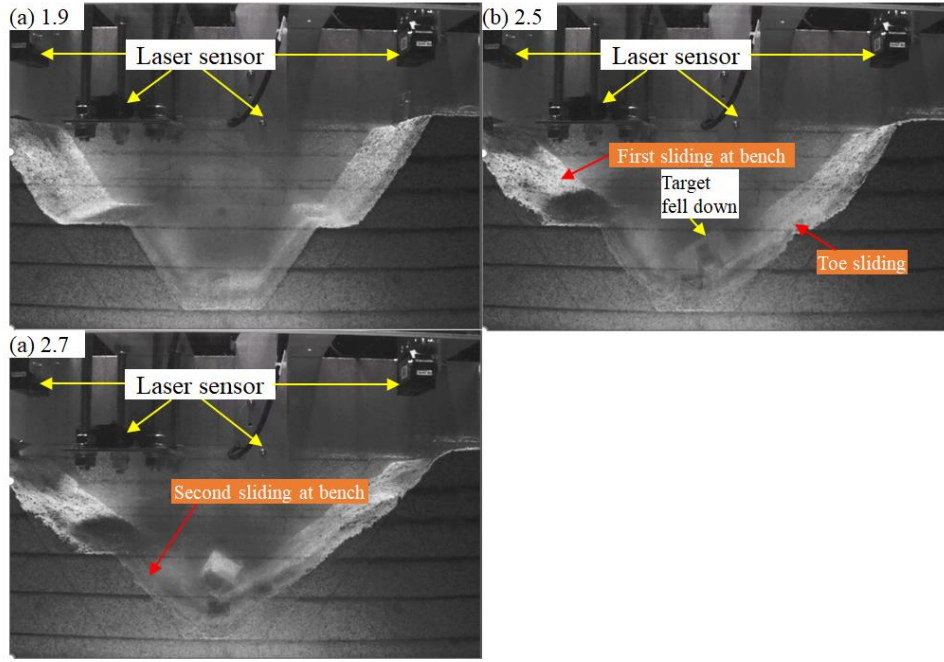


Fig. 7-36 Reaction of the model slopes to each shaking in Case 6 of the first group: (a) stable state after the first shaking; (b) first sliding at bench of the left slope and right toe sliding (c) second sliding at bench on the left slope

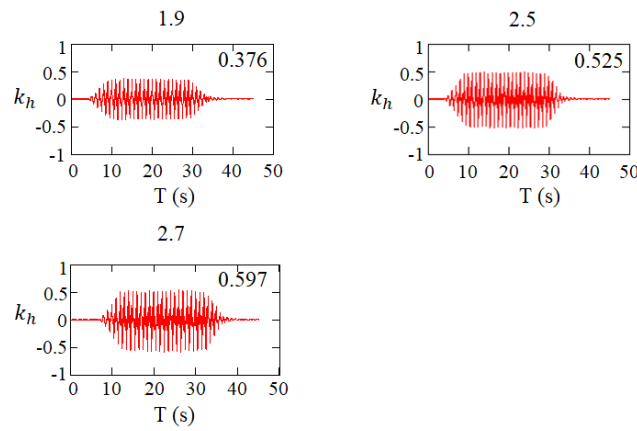


Fig. 7-37 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 6 of the first group

Displacement

In **Fig. 7-38**, the symbols can refer to those in **Fig. 7-13**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 6 of the first group. During the second shaking, LD3 went out of range, and LD4 decreased due to the fall of the right target. In the third shaking, rotation of the left target led to an increase in LD1 and a reduction in LD2.

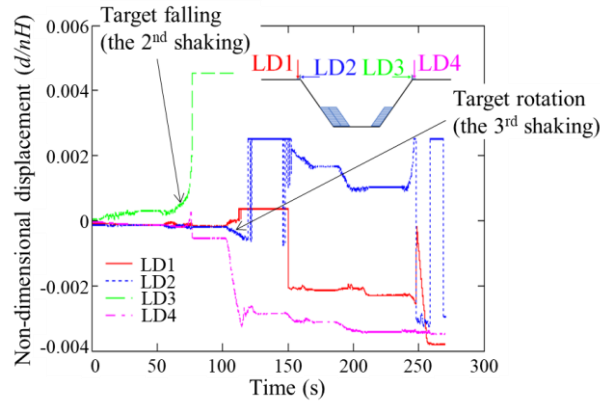


Fig. 7-38 Vertical and horizontal displacement of slope crests with each shaking in Case 6 of the first group

Earth pressure

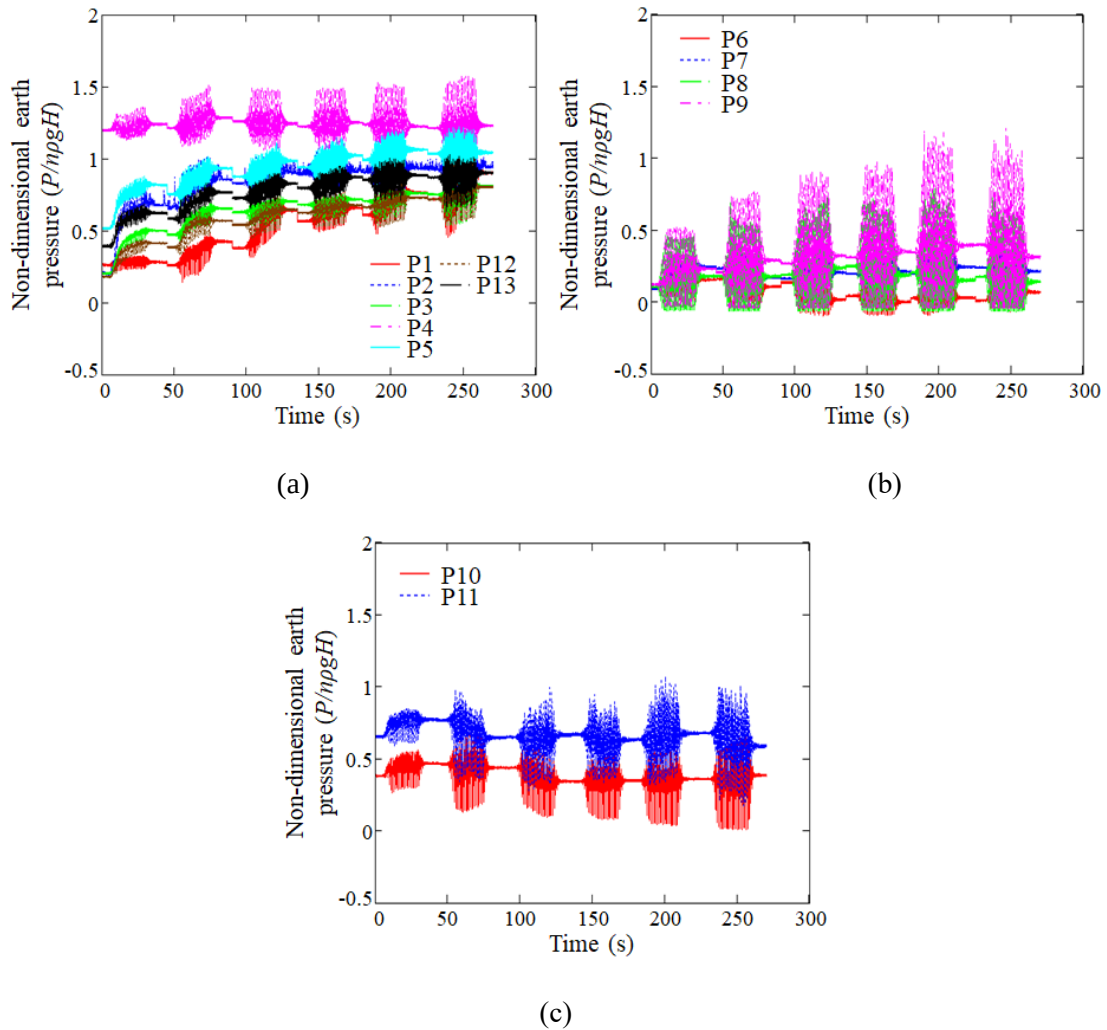


Fig. 7-39 Measured earth pressure with each shaking in Case 6 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Fig. 7-39 illustrates the variation of non-dimensional earth pressure with each shaking in Case 6, where the symbols can refer to those in **Fig. 7-14**. In this case, additional two earth pressure sensors (P12 and P13) were placed on the bottom of the model.

Acceleration

Fig. 7-40 displays the measured accelerations at various locations of the model slope during each shaking In Case 6 of the first group. The ACCT records the output wave from the shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

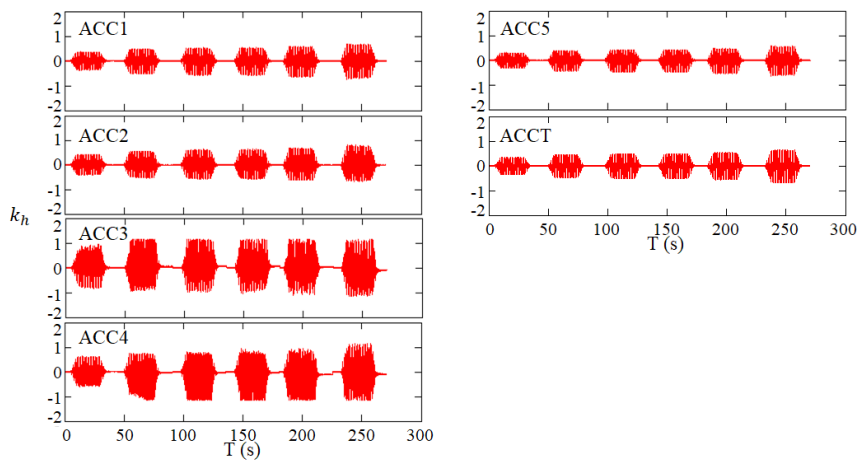


Fig. 7-40 Measured acceleration with each shaking in Case 6 of the first group

7.3.1.7 Case 7 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B_{HL}=3$ m, $B_{WL}=1.5$ cm, $B_{HR}=3$ m, $B_{WR}=1.5$ cm)

Sliding process

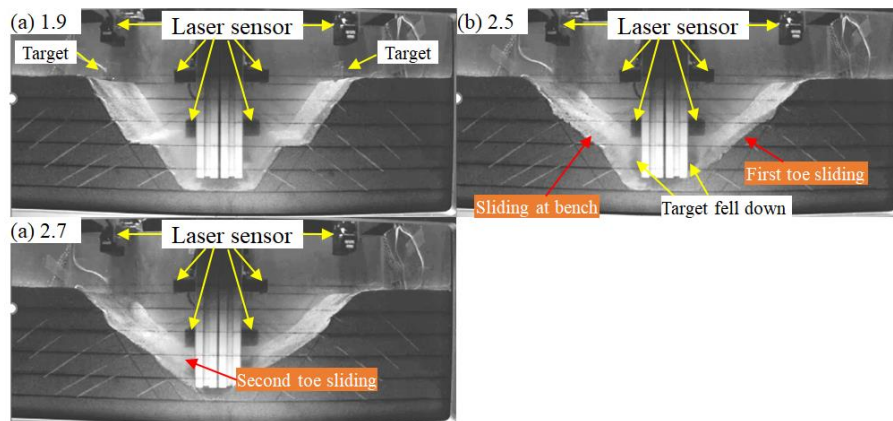


Fig. 7-41 Reaction of the model slopes to each shaking in Case 7 of the first group: (a) stable state after the first shaking; (b) sliding at bench of the left slope and right toe sliding; (c) toe sliding on the left slope

In Case 7 of the first group, both model slopes, which share the same geometric configuration, initially remained stable after the first shaking at a magnification of 1.9 ($k_h=0.365$) (see **Fig. 7-41 (a)**). However, during the second shaking, when the magnification reached 2.5 ($k_h=0.502$), the left second bench experienced a failure, and the right slope exhibited a toe sliding (see **Fig. 7-41 (b)**). This pattern continued in the third shaking at a magnification of 2.7 ($k_h=0.544$), where toe sliding subsequently occurred on the left slope (see **Fig. 7-41 (c)**).

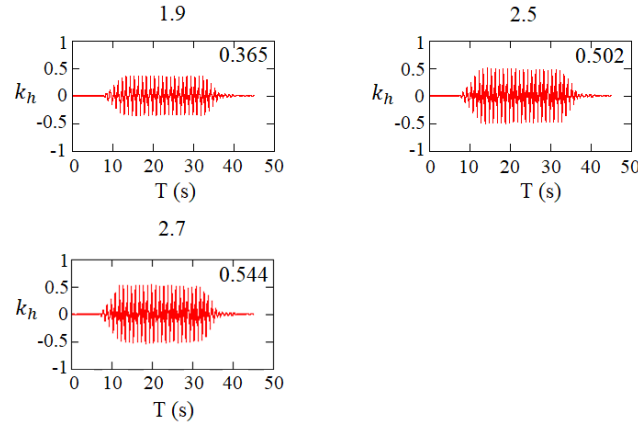


Fig. 7-42 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 7 of the first group

Displacement

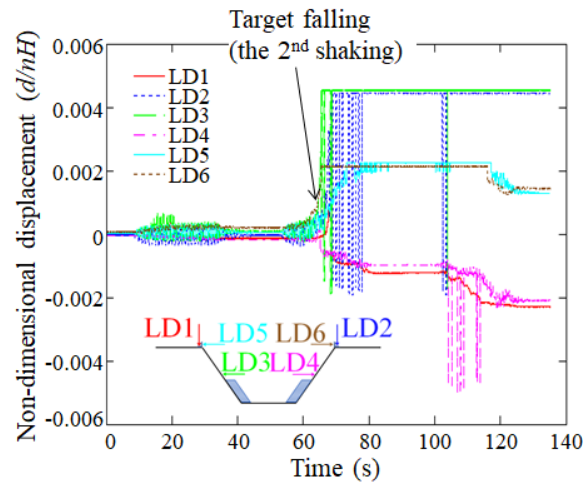


Fig. 7-43 Slope crests' vertical and horizontal displacement and horizontal displacement of slope surfaces with each shaking in Case 7 of the first group

The symbols in **Fig. 7-43** have the same meanings as those in **Fig. 7-13**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 7 of the first group.

Earth pressure

Fig. 7-44 shows change of non-dimensional earth pressure during each shaking in Case 7, where the symbols can refer to those in Fig. 7-14. In this case, two additional earth pressure sensors (P12 and P13) were placed on the model bottom.

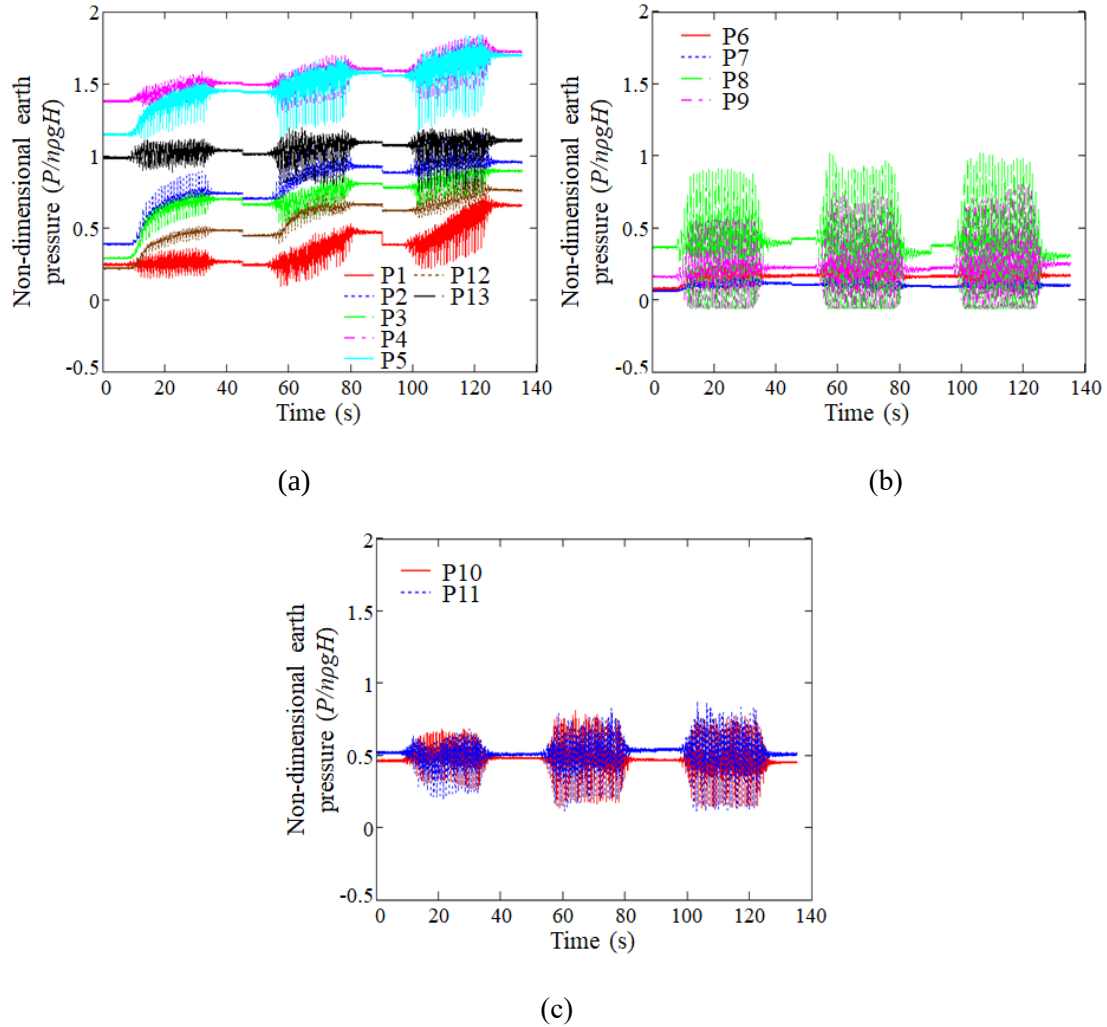


Fig. 7-44 Measured earth pressure with each shaking in Case 7 of the first group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 7-45 illustrates the measured accelerations at various locations of the model slope during each shaking in Case 7 of the first group. ACCT records the output wave from the shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

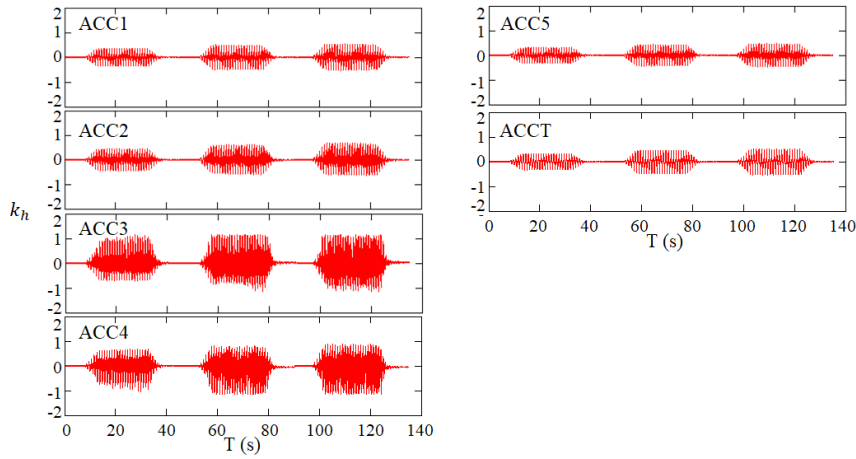


Fig. 7-45 Measured acceleration with each shaking in Case 7 of the first group

The experimental results for cases of the first group are summarized in **Table 7-7**. After the testing, the water content in the model slope was sampled from three distinct heights from the bottom of the model. The seismic coefficients when sliding happened on each bench are also presented.

Table 7-7 Experimental results of benched slopes with an additional bench

No.	Date	Final water content (%)			Sliding seismic coefficient			
		Bottom	Middle	Top	Left		Right	
		(0-2 cm)	(8-10 cm)	(16-18 cm)	upper	lower	upper	lower
1	220414	11.9	6.6	5.3	0.447 (2.5)	0.505 (2.7)	0.505 (2.7)	0.588 (3.0)
2	220415	11	7.4	7.2	Δ (1.9)	0.455 (2.5)	0.455 (2.5)	—
3	220425	9.7	7	5.9	0.566 (2.5)	0.566 (2.5)	0.566 (2.5)	0.624 (2.7)
4	220824	8.5	6.9	5.6	0.541 (2.7)	0.500 (2.5)	0.500 (2.5)	0.500 (2.5)
5	221003	9.9	7.6	5.9	0.502 (2.5)	0.502 (2.5)	0.502 (2.5)	0.502 (2.5)
6	221005	9.8	6.8	6	0.525 (2.5)	0.597 (2.7)	0.525 (2.5)	0.525 (2.5)
7	221019	11	7.1	5.2	0.502 (2.5)	0.544 (2.7)	0.502 (2.5)	0.502 (2.5)

7.3.2 Laterally confined benched slopes with equivalent excavation

For each case, the parameters β_L and β_R represent the left and right slope angles, respectively. The left first bench's height and width are denoted by $B1_{HL}$ and $B1_{WL}$, while those of the right first bench are represented by $B1_{HR}$ and $B1_{WR}$. Additionally, $B2_{HR}$ and $B2_{WR}$ refer to the height and width of the second bench on the right slope.

7.3.2.1 Case 1 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B1_{HL}=3$ m, $B1_{WL}=3$ cm, $B1_{HR}=1.5$ m, $B1_{WR}=1.5$ cm, $B2_{HR}=3$ m, $B2_{WR}=1.5$ cm)

Sliding process

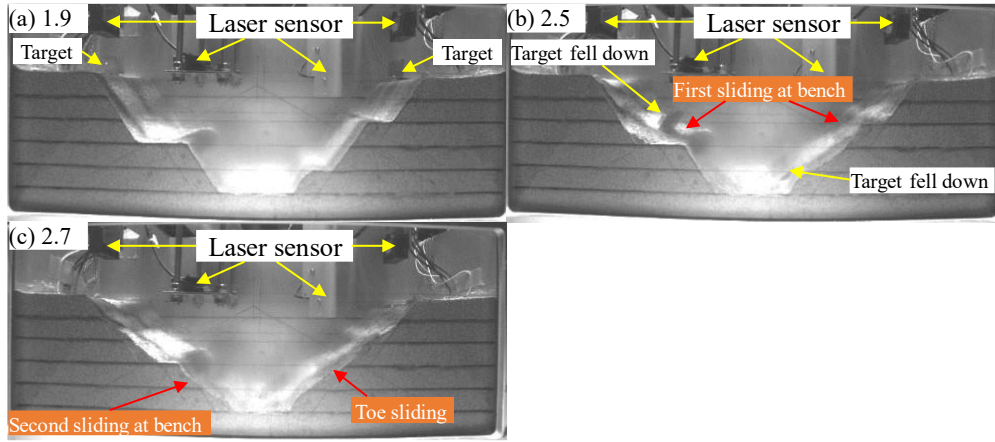


Fig. 7-46 Reaction of the model slopes to each shaking in Case 1 of the second group: (a) stable state after the first shaking; (b) sliding at bench on the left slope; (c) second sliding at bench on the left slope and toe sliding at the right slope

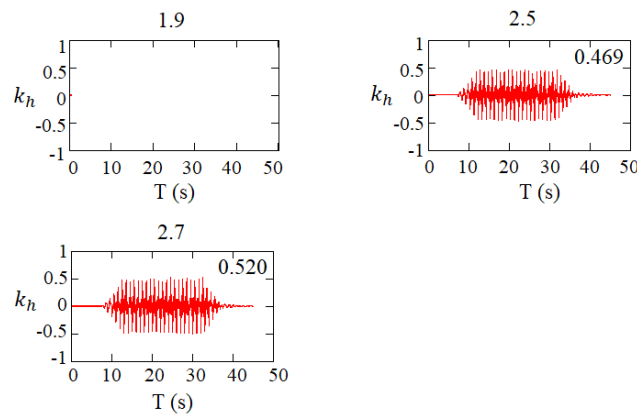


Fig. 7-47 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 1 of the second group

In this case 1, both slopes remained stable after the initial shaking with a magnification of 1.9 (see **Fig. 7-46 (a)**). However, data from this shaking were not recorded due to operator error. During the second shaking at a magnification of 2.5 ($k_h=0.469$), the left second bench failed, and a failure spanning the second and third benches of the right slope occurred (see **Fig. 7-46 (b)**). Subsequently, during the third shaking, the first bench of the left slope failed, and toe sliding was observed on the right slope (see **Fig. 7-46 (c)**).

Displacement

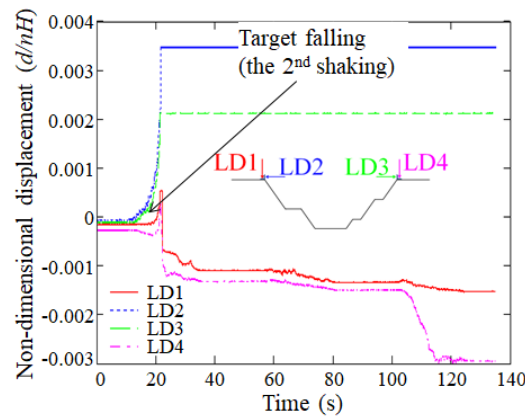
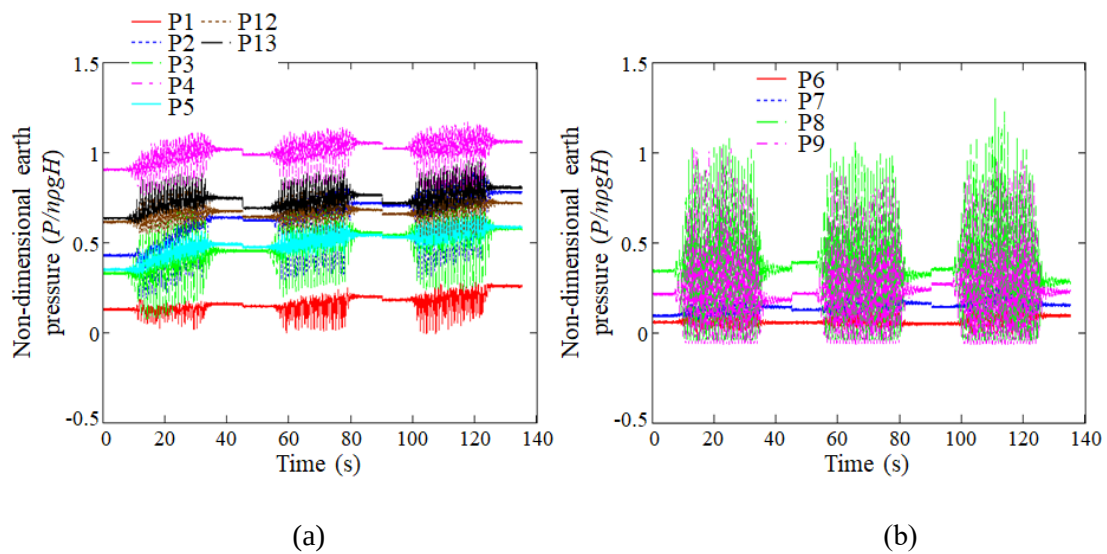


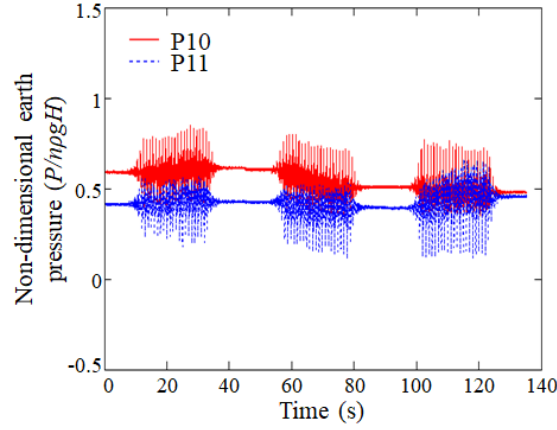
Fig. 7-48 Vertical and horizontal displacement of slope crests with each shaking in Case 1 of the second group

The symbols in **Fig. 7-48** have the same meanings as those in **Fig. 7-13**, illustrating the variation in non-dimensional displacement for each shaking in Case 1 of the second group. During the second shaking, both targets fell, causing LD2 and LD3 to went out of range and a reduction in readings for LD1 and LD4.

Earth pressure

Fig. 7-49 presents the variation in non-dimensional earth pressure with each shaking in Case 1 of the second group, where the symbols can refer to those in **Fig. 7-14**. In this case, two additional earth pressure sensors (P12 and P13) were placed on the model bottom.





(c)

Fig. 7-49 Measured earth pressure with each shaking in Case 1 of the second group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 7-50 illustrates the measured accelerations at various locations of the model slope during each shaking event. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The phenomenon of seismic coefficient amplification is evident, with accelerations increasing at higher elevations.

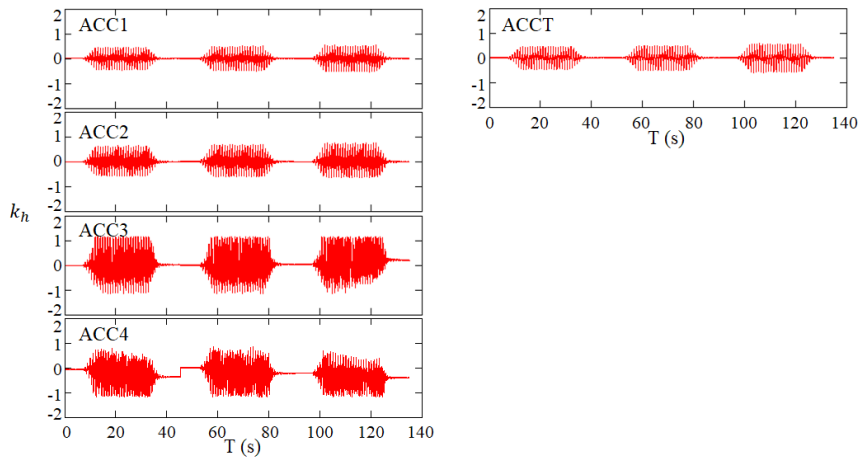


Fig. 7-50 Measured acceleration with each shaking in Case 1 of the second group

7.3.2.2 Case 2 ($\beta_L=55^\circ$, $\beta_R=55^\circ$, $B1_{HL}=3$ m, $B1_{WL}=3$ cm, $B1_{HR}=1.5$ m, $B1_{WR}=1.5$ cm, $B2_{HR}=3$ m, $B2_{WR}=1.5$ cm)

Sliding process

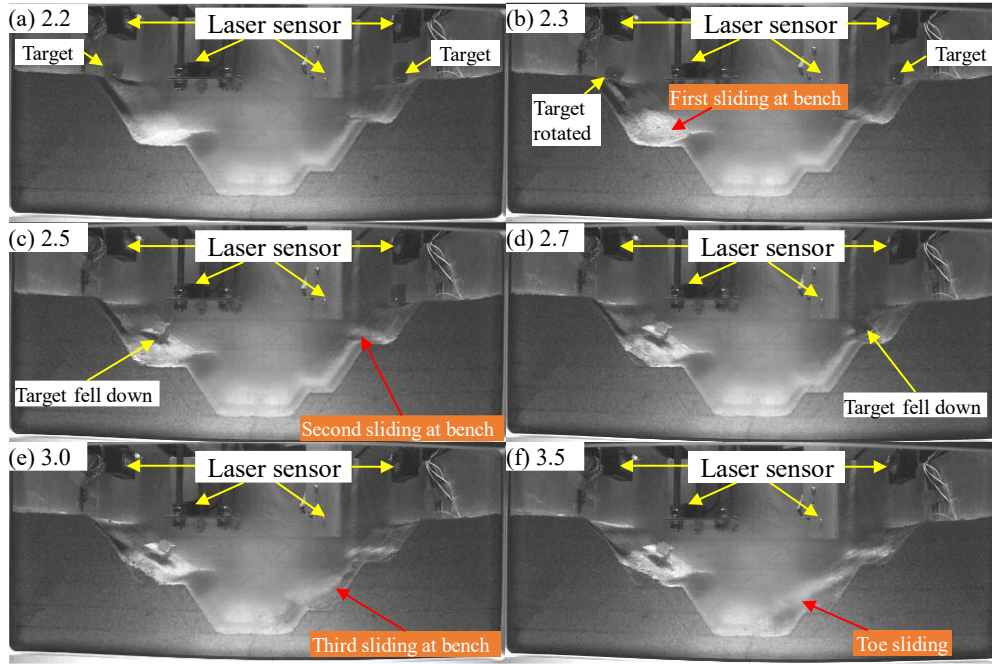


Fig. 7-51 Reaction of the model slopes to each shaking in Case 2 of the second group: (a) stable state after the first shaking; (b) sliding at bench on the left slope; (c) second sliding at bench on the right slope; (d) right target fell down; (e) third sliding at bench on the right slope; (f) toe sliding on the right slope

Case 2 of the second group is a repeated case of Case1 of the second group. Both slopes remained stable at the first shaking with a magnification of 2.2 ($k_h=0.420$) (see **Fig. 7-51 (a)**). During the second shaking with a magnification of 2.3 ($k_h=0.444$), a toe failure occurred at the left second bench. The debris slid from the center of the slope crest and accumulated in a fan-like shape on the top of the first bench (see **Fig. 7-51 (b)**). The reason for selecting different magnifications for the first two shakings, compared to those used in Case 1 of the second group, was to observe the varying seismic resistance of the two sides of the slopes. In contrast, in Case 1, both sides failed simultaneously.

At the third shaking with a magnification of 2.5 ($k_h=0.487$), the failure on the left slope developed and a slight local sliding also occurred on the crest of the right slope near the back wall (see **Fig. 7-51 (c)**). At the next ground motion with a magnification of 2.7 ($k_h=0.552$), the target on the right slope slid down while most of the slope was still stable (see **Fig. 7-51 (d)**). Then, a toe sliding across the right slope crest and the second bench was triggered by the shaking with a magnification of 3.0 ($k_h=0.643$) (see **Fig. 7-51 (e)**). This failure was located in the center of the right slope because the skirts of the benches remained intact. The first bench of the left slope did not fail during the test, while the first bench of the right slope was destroyed by the last shaking with a magnification of 3.5 ($k_h=0.892$) (see **Fig. 7-51 (f)**).

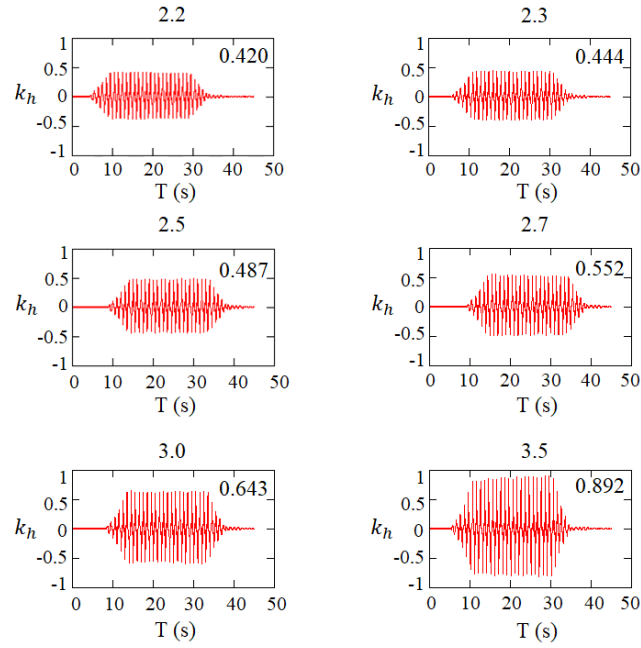


Fig. 7-52 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 2 of the second group

Displacement

The symbols in **Fig. 7-53** can refer to those in **Fig. 7-13**, illustrating the variation in non-dimensional displacement for each shaking in Case 2 of the second group. During the second shaking, the left target rotated, causing a rise in LD2 and a reduction in LD1. The left target then fell during the third shaking, causing LD2 to go out of range and further decreasing LD1. During the fourth shaking, the right target fell, leading to LD3 moving out of range and a decrease in LD4.

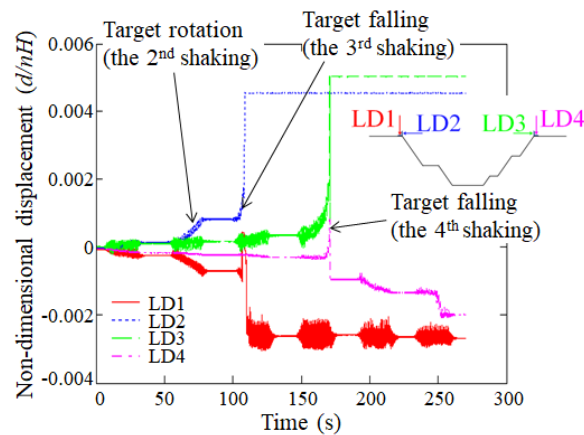


Fig. 7-53 Vertical and horizontal displacement of slope crests with each shaking in Case 2 of the second group

Earth pressure

Fig. 7-54 presents change of non-dimensional earth pressure with each shaking in Case 2 of the second group, where the symbols can refer to those in **Fig. 7-14**. In this case, two additional earth pressure sensors (P12 and P13) were placed on the model bottom.

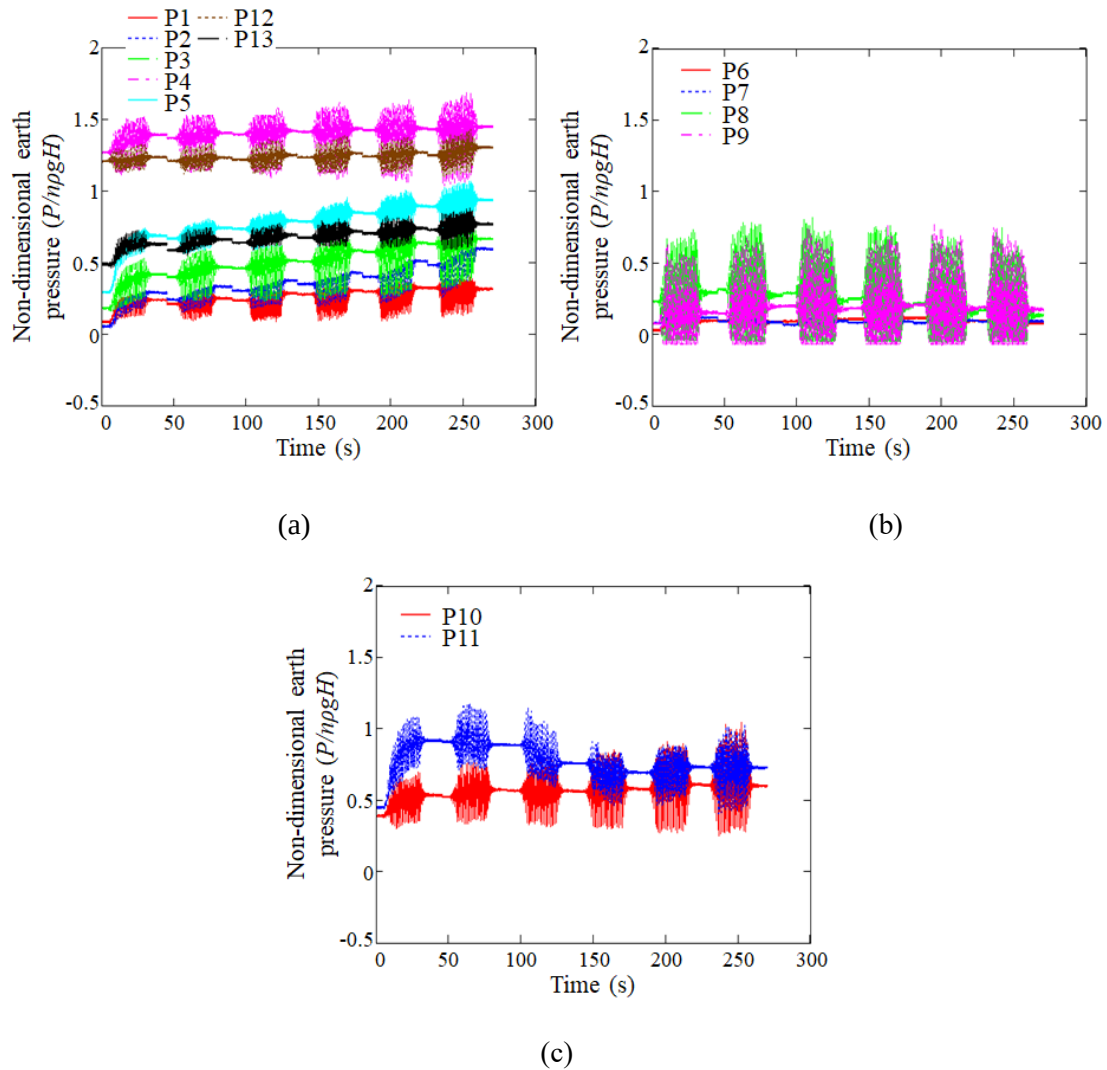


Fig. 7-54 Measured earth pressure with each shaking in Case 2 of the second group: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 7-55 illustrates the measured accelerations at various locations of the model slope during each shaking event. ACCT records the output wave of the shaking table. The acceleration values have been converted to the prototype scale k_h by dividing by 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations. ACC1 malfunctioned in this case.

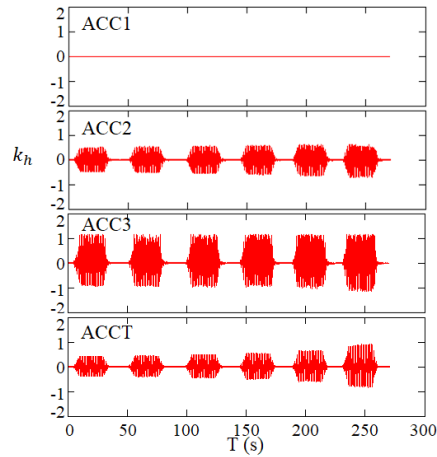


Fig. 7-55 Measured acceleration with each shaking in Case 2 of the second group

The experimental results for the cases in the first group are summarized in **Table 7-8**. After the experiment, water content in the model slope was measured by sampling at three distinct heights from the bottom of the model. The seismic coefficients at which sliding occurred on each bench are also provided.

Table 7-8 Experimental results of benched slopes with equivalent excavation

No.	Date	Final water content (%)			Sliding seismic coefficient				
		Bottom	Middle	Top	Left		Right		
		(0-2 cm)	(8-10 cm)	(16-18 cm)	first	second	first	second	third
1	230405	11.9	6.6	5.3	0.469 (2.5)	0.469 (2.5)	0.520 (2.7)	0.469 (2.5)	0.469 (2.5)
2	230721	11	7.4	7.2	—	0.444 (2.3)	0.892 (3.5)	0.643 (3.0)	0.487 (2.5)

7.4 Discussion

7.4.1 Stability and failure modes of benched slopes in the test

7.4.1.1 First group of benched slopes with an additional bench

With the addition of a first bench, the failure mode differs compared to the uniform slopes examined in **Chapter 6**. As observed in both slopes of Case 1, both slopes of Case 2, the right slope of Case 3, the left slope of Case 6, and the left slope of Case 7, the initial sliding generally occurred on the second bench while the first bench remained stable. Accelerations monitored in the model confirmed a pronounced seismic amplification effect in the dynamic cases, resulting in larger inertial forces acting on the upper portions of the benched slope.

Comparing Case 1, which features higher first benches, with Case 2 and Case 3, which have lower first benches, the slopes in Case 1 exhibited greater stability. Given that the primary slope structure behind the first bench was identical across these three cases, the first bench can be regarded as a counterweight for the toe. A higher, heavier bench provided greater resisting force to the slope section behind it. In Case 4, where the left slope had the tallest bench among all cases, enhanced stability was

observed at the slope crest; however, the first bench failed during the initial shaking. In Case 6, where the left bench width was double that of the right bench, the increased bench width did not contribute to slope stability, as failures occurred simultaneously on both the first and second benches.

Generally, a higher first bench contributes positively to dynamic slope stability; however, if the bench is excessively high, sliding may shift to the first bench itself. In contrast, increasing the width of the first bench does not significantly enhance dynamic stability.

Case 5 demonstrates the effect of lateral resistance from the first bench. The results suggest that this resistance can be disregarded when assessing the safety of the benched slope, as the lateral area of the first bench is relatively small compared to the slope's overall lateral surface.

7.4.1.2 Second group of benched slopes with equivalent excavation

In both cases within this group, the initial sliding occurred consistently at the bench level. In the first case, failure occurred on both sides during the second shaking at a magnification of 2.5. Sliding took place on the left second bench and on the second and third benches of the right slope. Less soil slid down from the right crest than from the left crest.

Since both slopes failed during the second shaking in the first case, differences between the two benched slope shapes could not be observed. In the second case, a smaller magnification of 2.3 was applied for the second shaking, resulting in failure of the left slope with two benches before the right slope with three benches. These results indicate that the right slope exhibited greater resistance to ground motions than the left slope. This suggests that a rotational failure mechanism dominated the benched slope, rather than a translational failure mechanism, as both slopes had the same soil weight and differed only in shape. Therefore, with equivalent excavation, more benches of lower height are preferable. The initial sliding surface followed a three-dimensional curved path, consistent with the previously proposed mechanism for slopes with lateral restraint (Michalowski & Drescher, 2009).

7.4.2 Overburden pressure at the bottom of the model

In each case of the first group, up to seven earth pressure sensors (P1 to P5, P12, and P13) were positioned along the longitudinal centerline at the base of the model to measure variations in overburden pressure during ground motions. Sensor P1 was located beneath the center of the model, while P2 and P13 were positioned below the slope surface. Sensors P3 and P12 were placed vertically below the slope toes, irrespective of the first bench. Finally, P4 and P5 were located vertically below the slope crests.

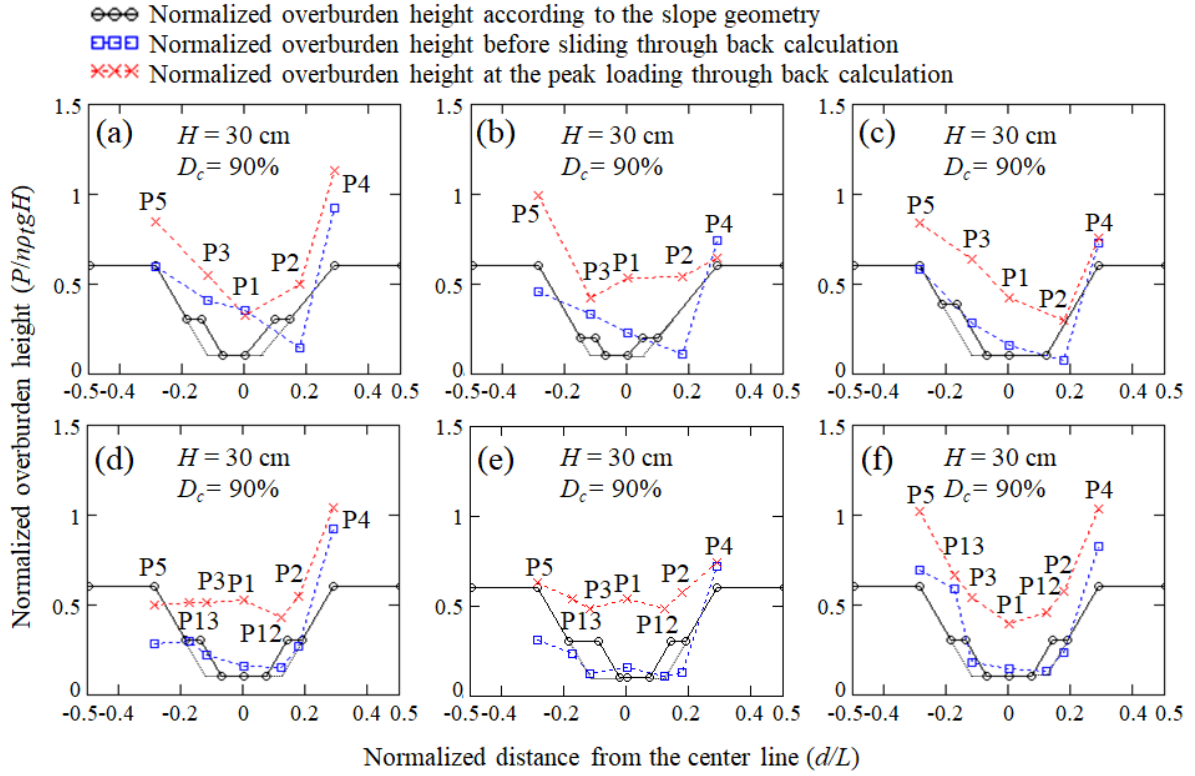


Fig. 7-56 Normalized overburden height at bottom of the model of the first group before and after seismic loading: (a) Case 1; (b) Case 3; (c) Case 4; (d) Case 5; (e) Case 6; (f) Case 7

Fig. 7-56 compares normalized overburden height, calculated through back analysis, before and after seismic loading with the normalized actual overburden height. Data from Case 2 are excluded due to an operational error during testing. The results indicate that earth pressure increased following shaking in all cases.

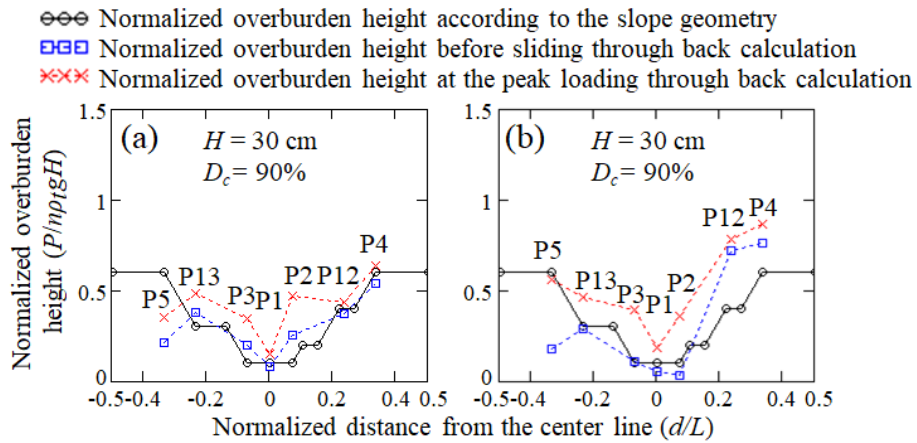


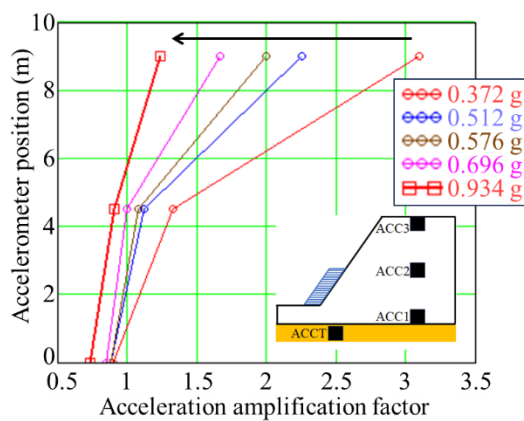
Fig. 7-57 Normalized overburden height at bottom of the model of the second group before and after seismic loading: (a) Case 1; (b) Case 2

The configuration of earth pressure sensors on the model base in the second group was slightly modified (see **Fig. 7-57**). Sensor P1 was positioned at the center of the model, while P2 and P3 were located vertically below the slope toes. Sensors P12 and P13 were placed beneath the slope surface.

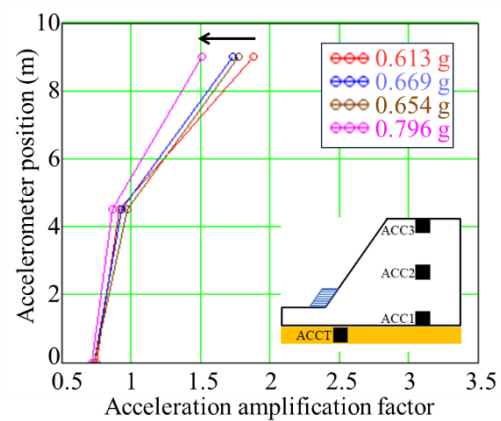
7.4.3 Seismic amplification with increasing magnitude of ground motions

The concept of the AAF, explained in **Section 6.4.4**, illustrates the effect of elevation amplification (EAE) in a slope subjected to ground motions. The measured results are presented below.

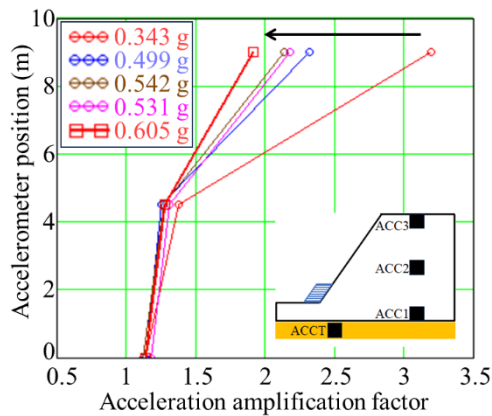
7.4.3.1 First group of benched slopes with an additional bench



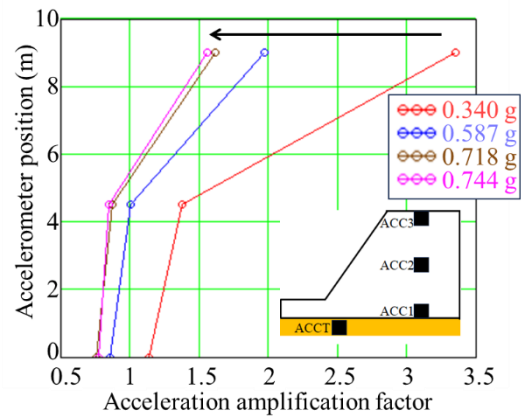
(a)



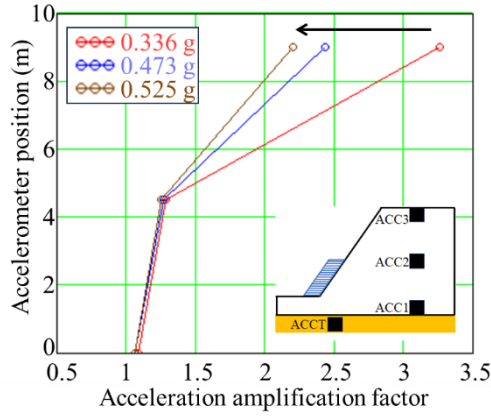
(b)



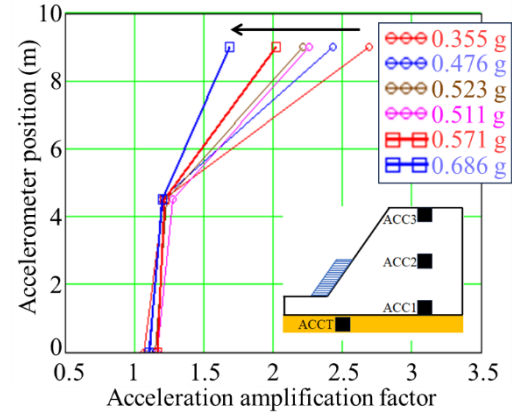
(c)



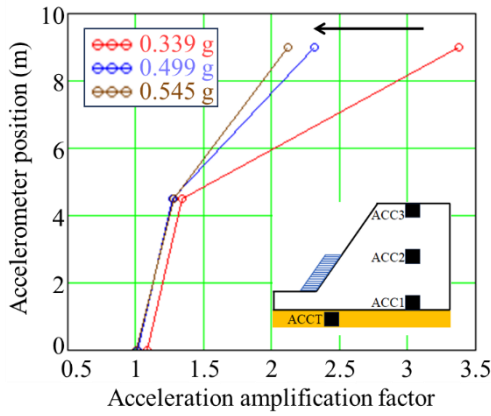
(d)



(e)



(f)



(g)

Fig. 7-58 AAF of benched slopes of the first group: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7

Fig. 7-58 illustrates elevation amplification effect across the seven cases in the first group. In Case 2, the recorded accelerations of the shaking table are irregular due to incorrectly set coefficients. Acceleration increases in a nonlinear pattern with height, with amplification levels varying based on the magnitude of the input wave. At the slope's midpoint (ACC2), changes in acceleration are minimal, whereas at the top (ACC3), the variation is pronounced. The acceleration amplification factor (AAF) at the top of the slope reduces as the input wave magnitude rises. The first bench's presence does not influence amplification within the slope's interior.

The calculation method of Arias intensity was introduced in **Section 6.4.4**. For the cases of the first group, **Fig. 7-59** to **Fig. 7-65** illustrate the evolution of Arias intensity at various heights on the model slope as shaking intensity increases. In Case 1, the final steady stage of Arias intensity for ACC3 is absent (see **Fig. 7-59**) because the accelerometer was displaced by the last strong shaking.

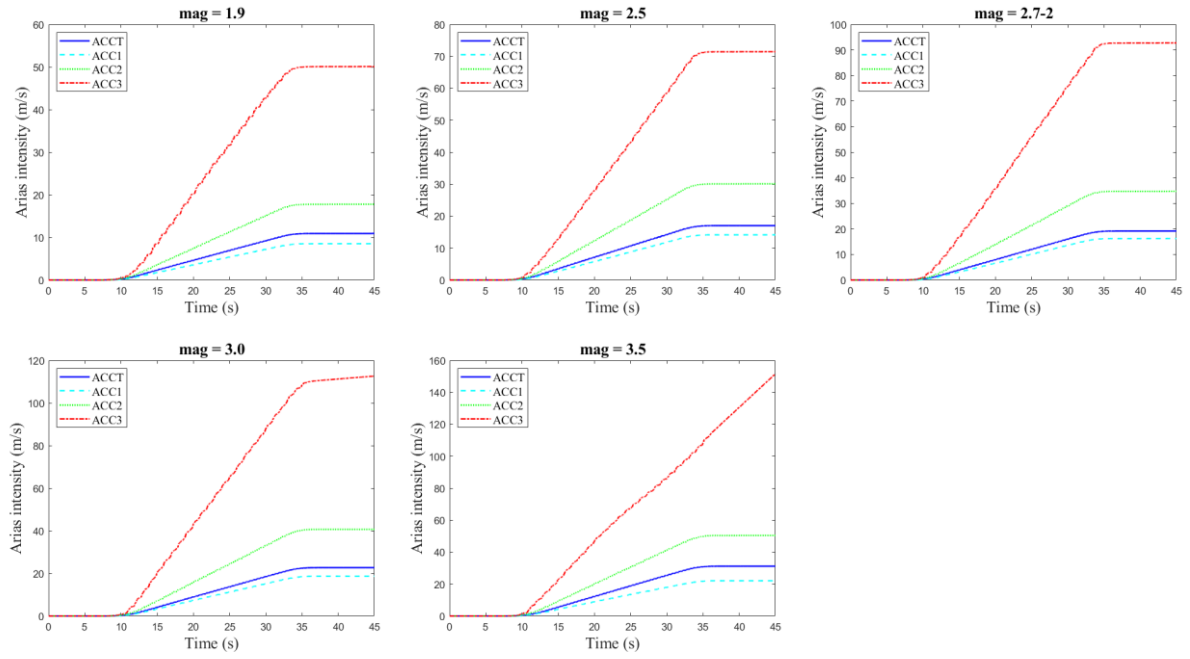


Fig. 7-59 Arias intensity evolution at various heights in Case 1 of the first group

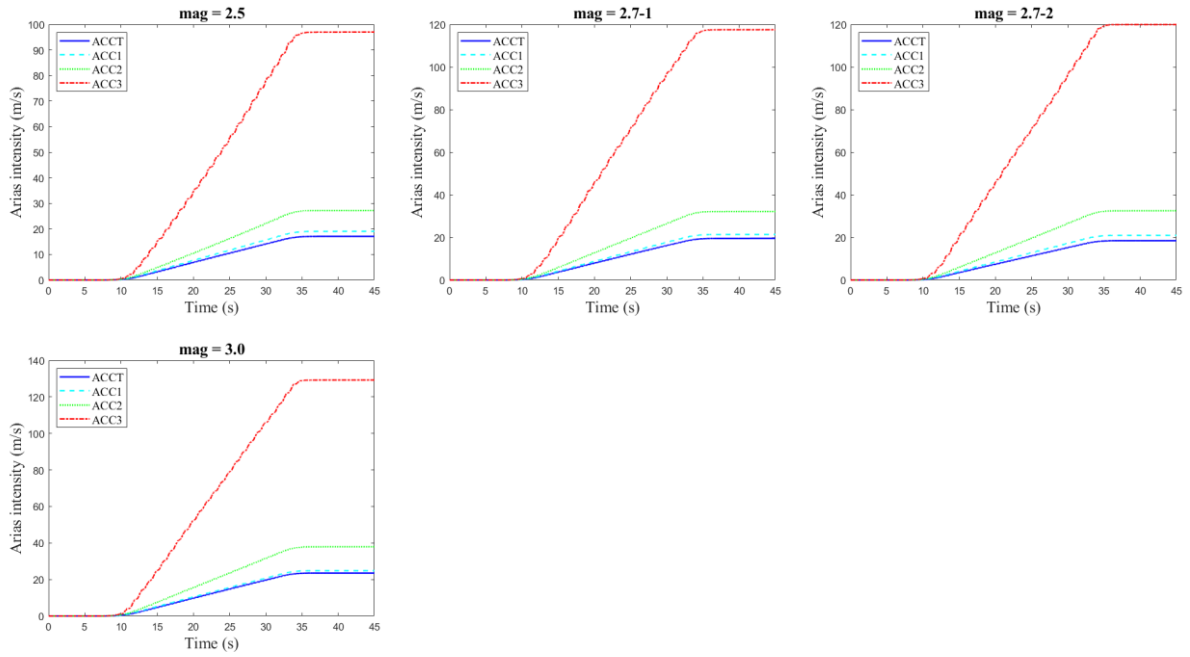


Fig. 7-60 Arias intensity evolution at various heights in Case 2 of the first group

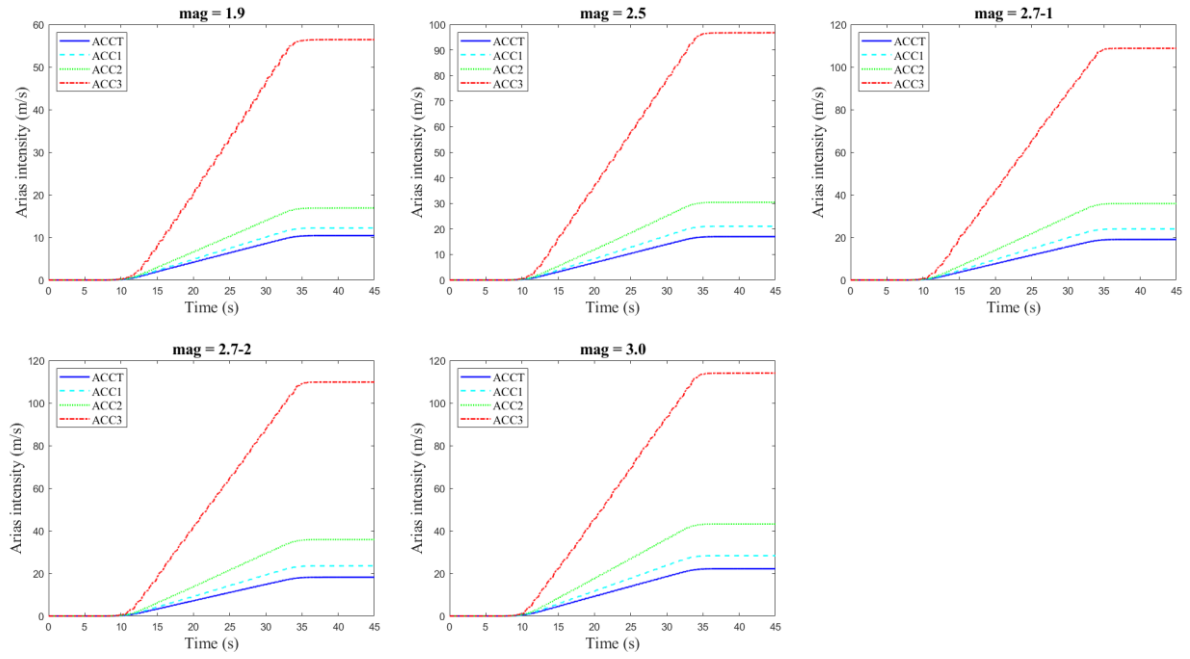


Fig. 7-61 Arias intensity evolution at various heights in Case 3 of the first group

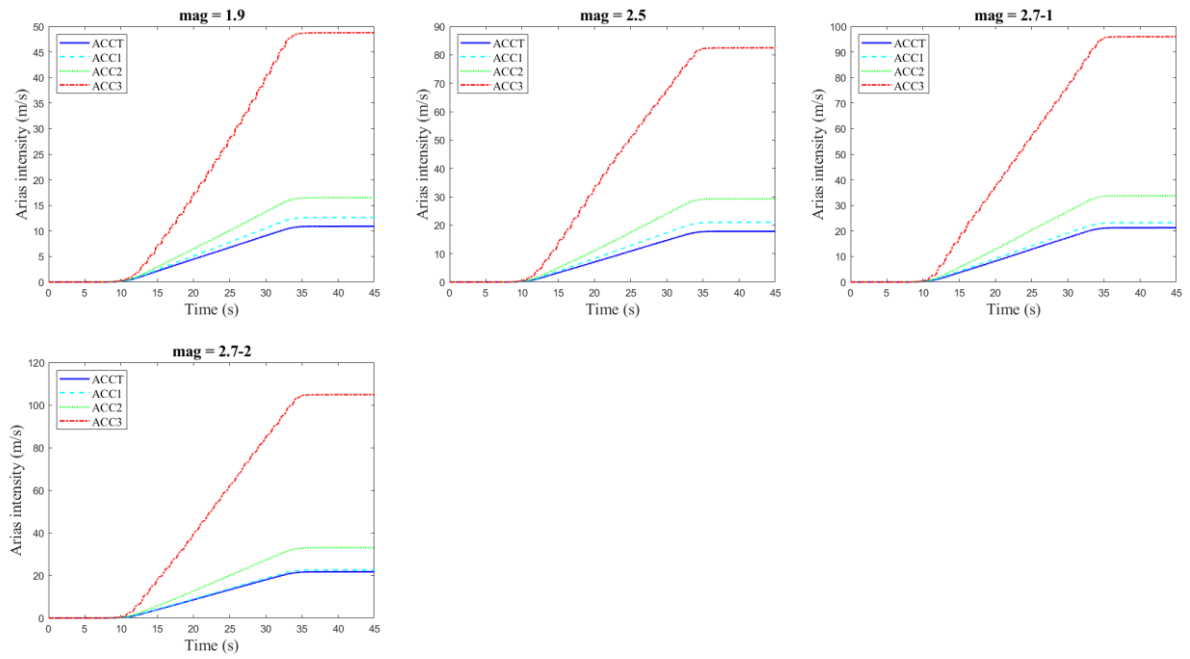


Fig. 7-62 Arias intensity evolution at various heights in Case 4 of the first group

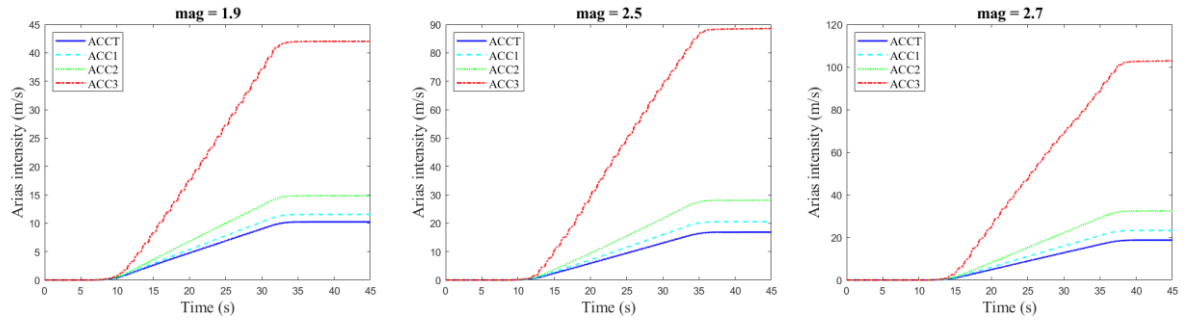


Fig. 7-63 Arias intensity evolution at various heights in Case 5 of the first group

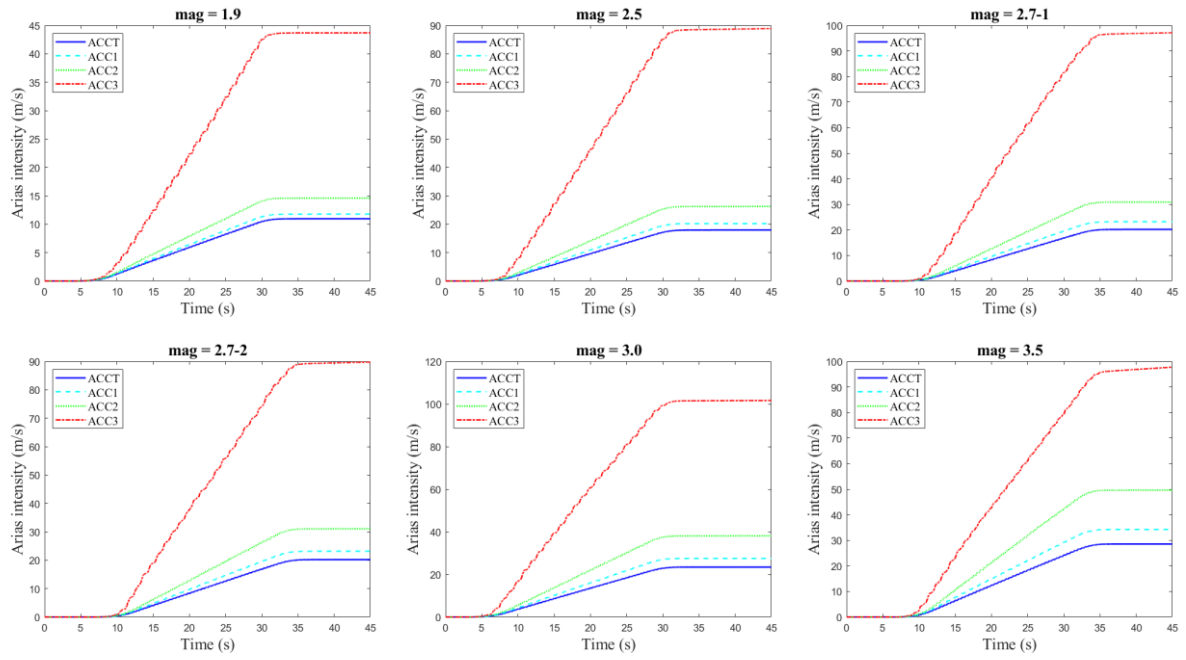


Fig. 7-64 Arias intensity evolution at various heights in Case 6 of the first group

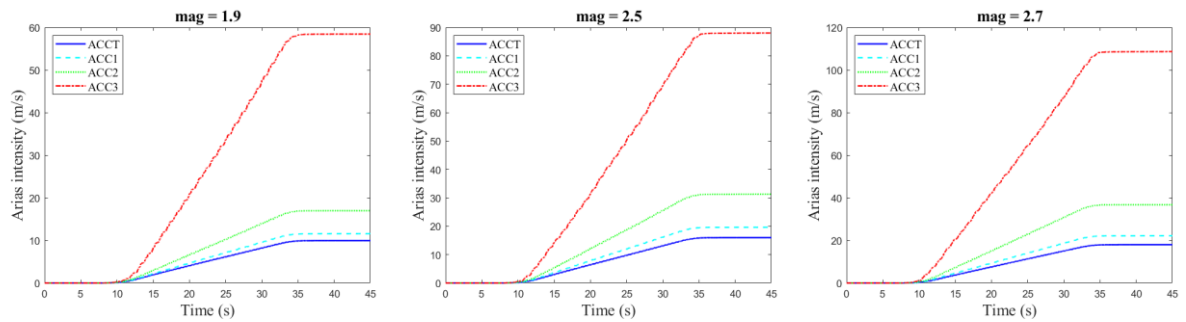


Fig. 7-65 Arias intensity evolution at various heights in Case 7 of the first group

7.4.3.2 Second group of benched slopes with equivalent excavation

In Case 2 of the second group, ACC1 malfunctioned, the peak values of ACC1 are set to be the same as ACCT (see Fig. 7-66). The same trend of seismic amplification with increasing magnitude of input waves as the first group can be observed.

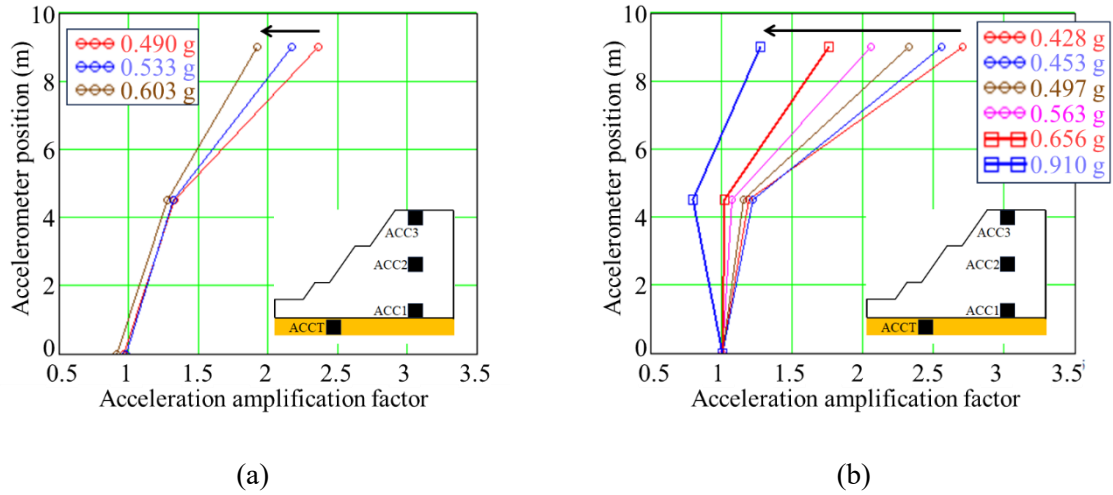


Fig. 7-66 AAF of the benched slope of the second group: (a) Case 1; (b) Case 2

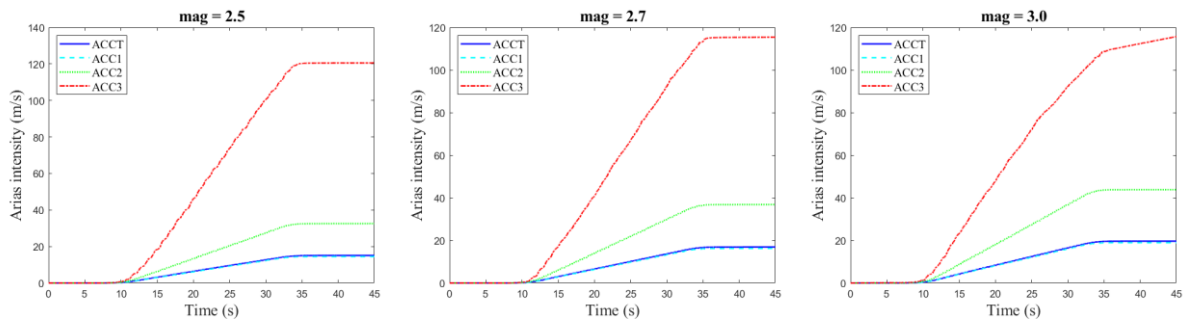
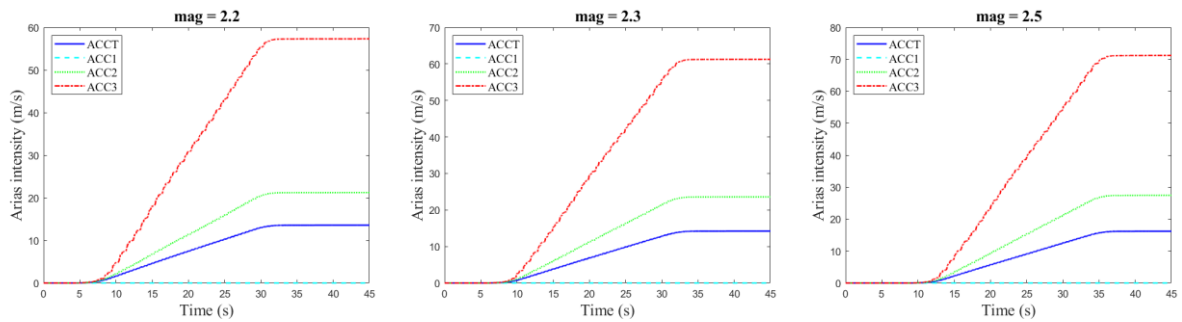


Fig. 7-67 Arias intensity evolution at various heights in Case 1 of the second group



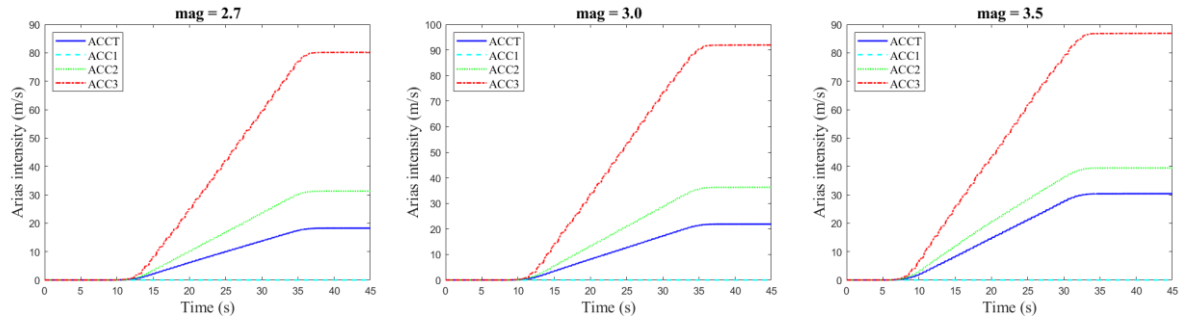


Fig. 7-68 Arias intensity evolution at various heights in Case 2 of the second group

For cases of the second group, **Fig. 7-67** and **Fig. 7-68** depict the evolution of Arias intensity at different height of the model with intensity of shaking. In Case 2, ACC1 malfunctioned and its Arias intensity becomes zero.

7.4.4 Boundary effect caused by the rigid container

Boundary effects are crucial to determine the seismic response of models. A rigid box made of aluminum alloy was used as the soil chamber instead of a laminar shear box, owing to its lower cost and ease of fabrication. No buffer material was placed between the model slope and the box walls, as geofoam has limited load-bearing capacity. However, input ground motions can produce boundary reflection waves within the rigid soil chamber, restricting horizontal deformation (Wang & Li, 2021). Therefore, validating the applicability of the rigid soil chamber is essential by comparing slope models with fully reflected boundaries to those with free boundaries.

A symmetric embankment-shaped benched slop model was constructed at the center of the soil chamber, as shown in **Fig. 7-69**. This model served as a comparison to Case 7 of the first group, sharing the same slope height, slope angle, and bench geometry. In this case, the two model slopes from Case 7 were combined to form the embankment. The sensor configuration within the model is presented in **Fig. 7-70**, with five accelerometers positioned vertically along the centerline of the model.

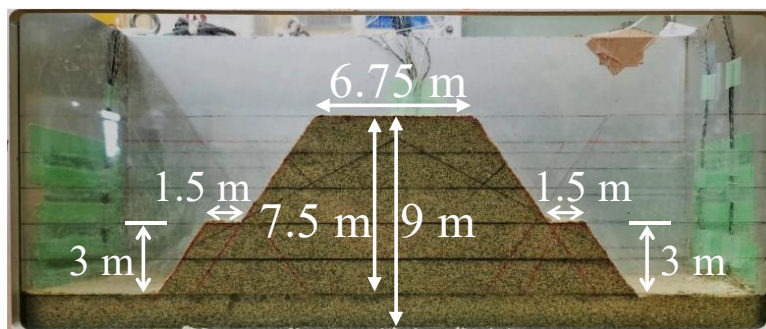


Fig. 7-69 Benched slopes without the reflected boundary (prototype scale)

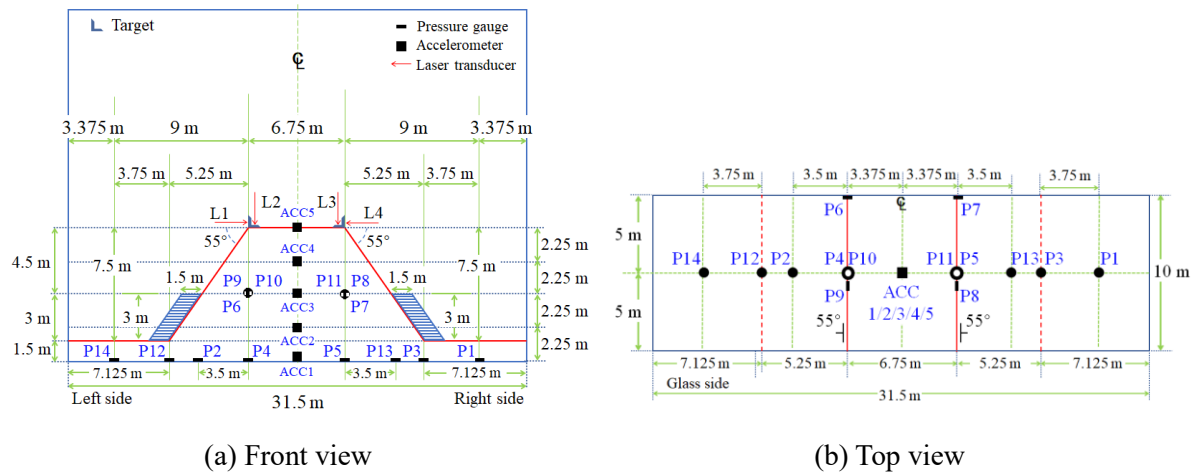


Fig. 7-70 Configuration of sensors in the embankment-shaped benched slope model

During the test, two seismic excitations were exerted with magnifications of 1.9 and 2.5. In the first shaking, both model slopes experienced sliding on the second bench, followed by toe sliding in the second shaking (see **Fig. 7-71**). This model appeared less stable than the model in Case 7 of the first group, as failure occurred during the first shaking. However, the actual magnitude of the input shaking should be verified.

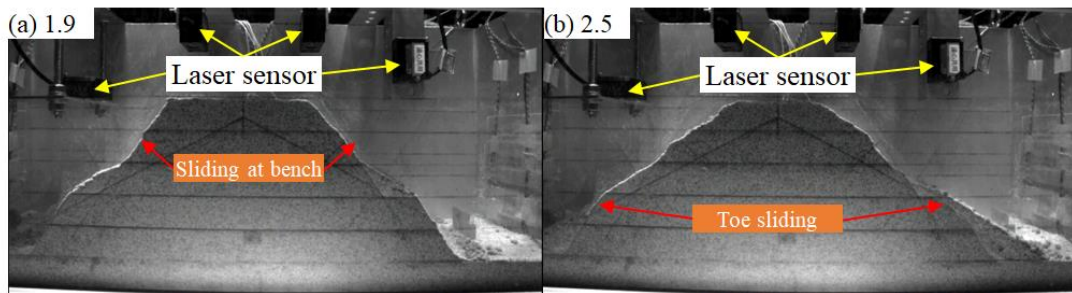


Fig. 7-71 Reaction of the model slopes to each shaking in the embankment-shaped model: (a) bench sliding on both sides; (b) toe sliding on both sides

Fig. 7-72 presents the measured acceleration of the model at various heights. The seismic coefficient for ACC1 in this case (0.419) was higher than that in Case 7 (0.365; see **Fig. 7-42**), indicating that it cannot be conclusively stated that the embankment-shaped benched slopes were inherently weaker.

Yang et al. (2004) suggested that the seismic response of models in flexible and rigid container converges as foundation densification increases. Wang and Li (2021) argued that the boundary effect was influenced by the ratio between the elastic modulus of the material and that of the container. In **Section 6.4.3**, the shear stress and strain was calculated, the shear modulus (G) of the damped sand was approximately 0.054 GPa. Given a Poisson's ratio (ν) of around 0.15 for dense sand (Thota et al., 2021). The Young's modulus (E) of the material can be obtained as $E=2G(1+\nu)$, resulting in a value of 0.124 GPa. In comparison, the E of the aluminum alloy is 80 GPa. Therefore, the ratio between the sand and

the container is 1.55×10^{-3} , which can result in a deviation (u_1) of 8% in the acceleration near the boundary relative to that at the center of the model.

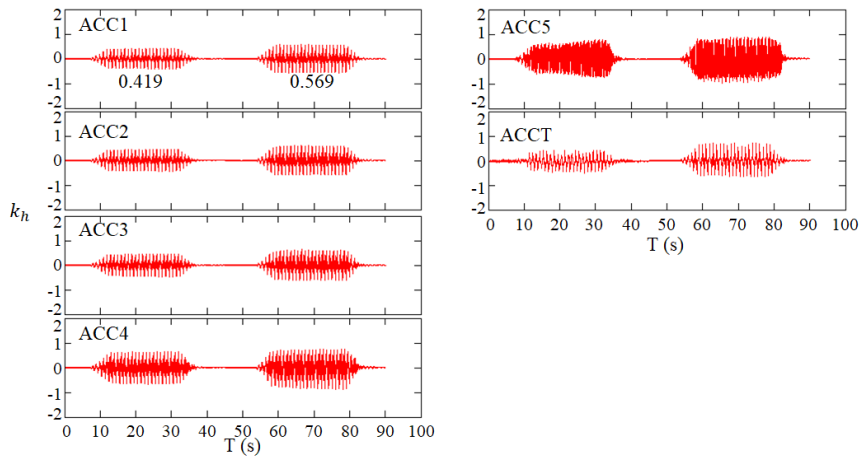


Fig. 7-72 Measured acceleration with each shaking in the embankment-shaped model

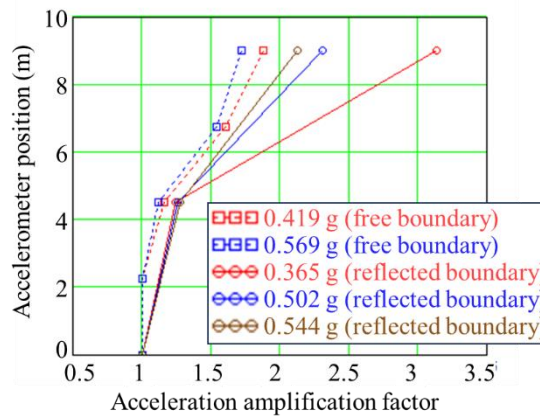


Fig. 7-73 Acceleration amplification of the benched slope with distinct boundary conditions

Using the peak value measured by ACC1 as a reference, the ratios of peak values at various heights are calculated and presented in **Fig. 7-73**. Acceleration amplification was less pronounced for the embankment-shaped model with a free boundary compared to slopes of Case 7 with a reflected boundary. This observation aligns with the results reported by Hung et al. (2018).

In conclusion, the results obtained with a rigid boundary can be considered a lower bound, as a larger seismic response may be induced under the same level of input waves. However, this deviation is acceptable for models constructed with dense sand.

7.4.5 Comparison between the experiments and the theory

For four failure modes, their variation of FS with respect to k_h can be calculated and compared. The curve that first decreases to 1 is considered the most critical, and based on this, the upper 80%

confidence interval is plotted. The results are presented below.

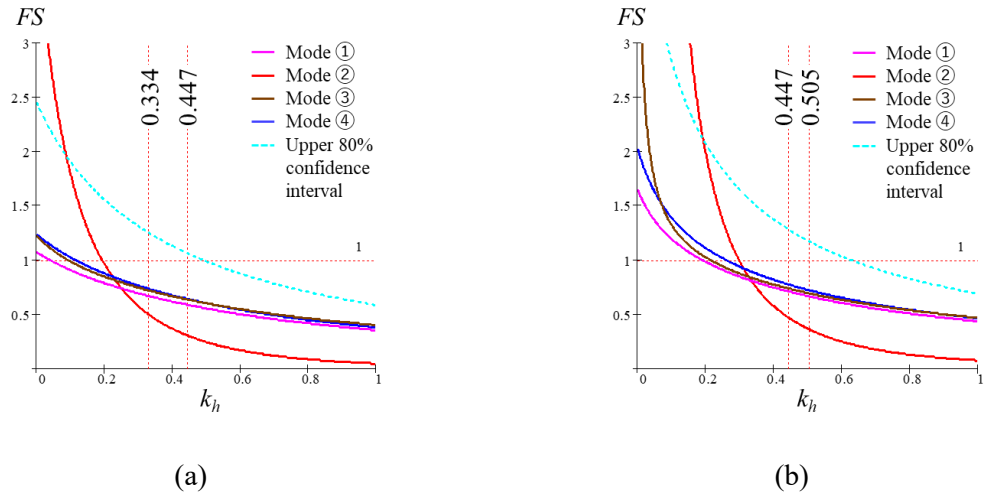


Fig. 7-74 Comparison between the prediction and the experiments of Case 1 of the first group: (a) left slope; (b) right slope

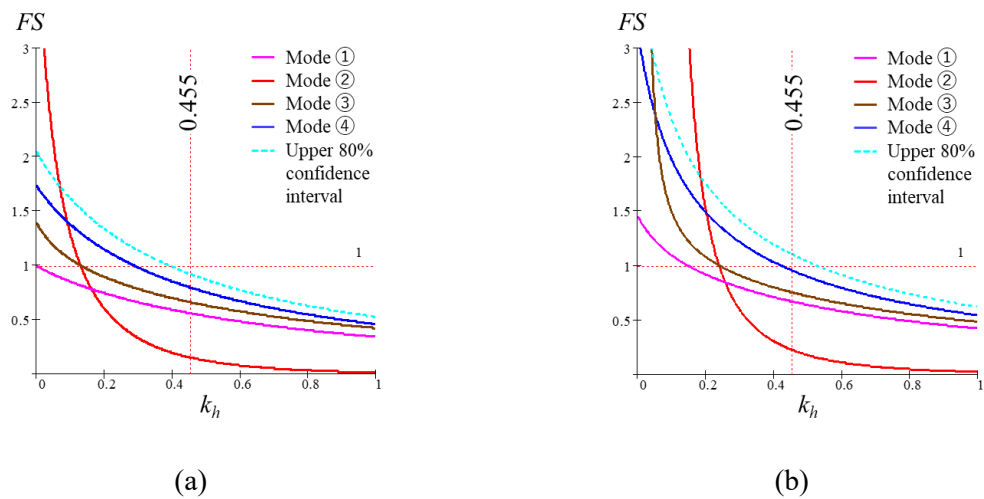
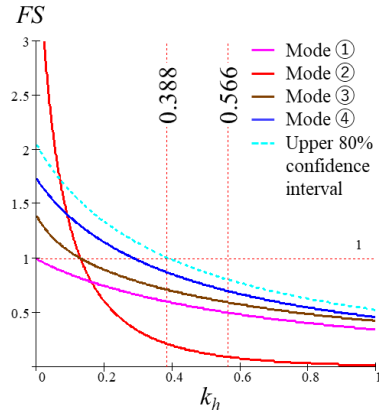
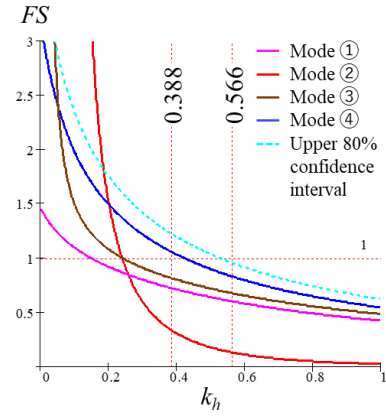


Fig. 7-75 Comparison between the prediction and the experiments of Case 2 of the first group: (a) left slope; (b) right slope

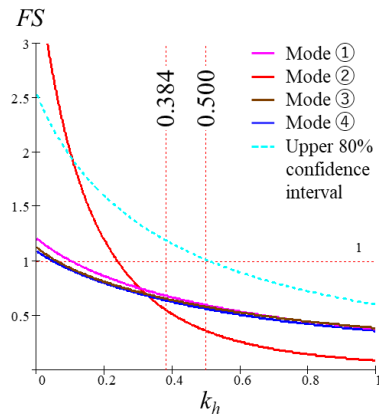


(a)

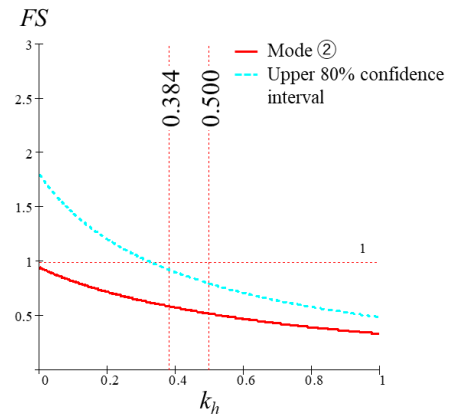


(b)

Fig. 7-76 Comparison between the prediction and the experiments of Case 3 of the first group: (a) left slope; (b) right slope

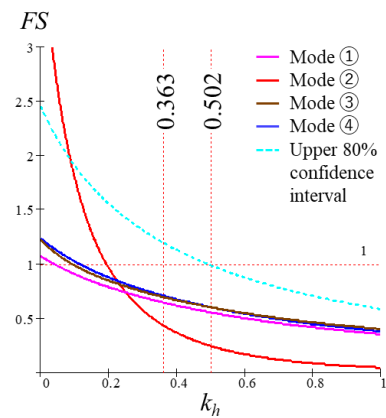
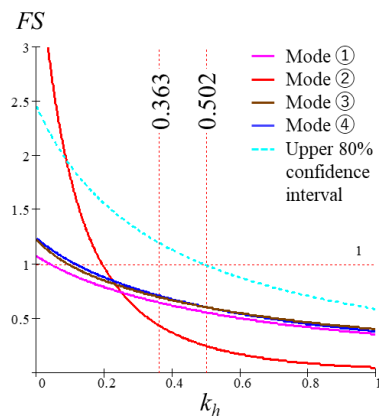


(a)



(b)

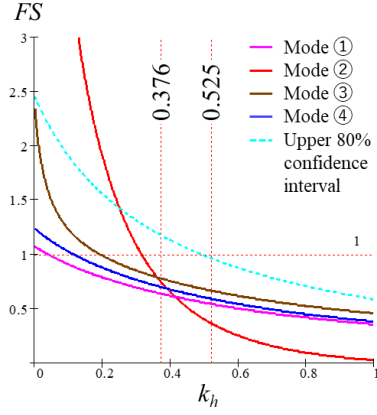
Fig. 7-77 Comparison between the prediction and the experiments of Case 4 of the first group: (a) left slope; (b) right slope



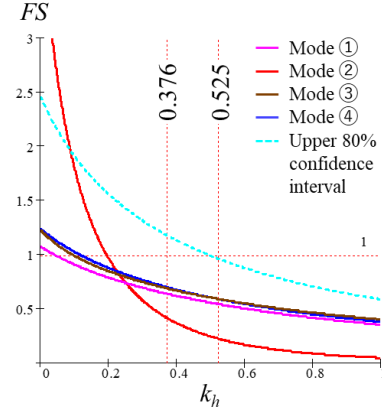
(a)

(b)

Fig. 7-78 Comparison between the prediction and the experiments of Case 5 of the first group: (a) left slope; (b) right slope

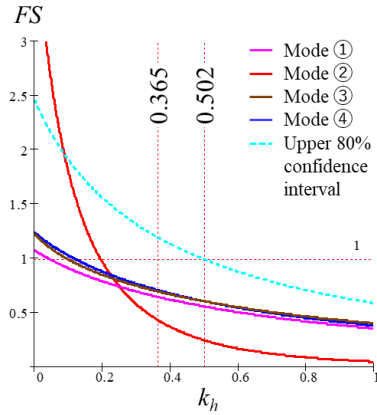


(a)

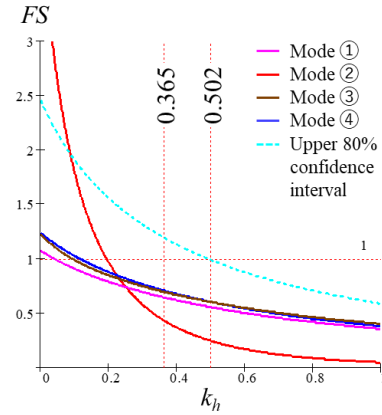


(b)

Fig. 7-79 Comparison between the prediction and the experiments of Case 6 of the first group: (a) left slope; (b) right slope



(a)



(b)

Fig. 7-80 Comparison between the prediction and the experiments of Case 7 of the first group: (a) left slope; (b) right slope

From **Fig. 7-74** to **Fig. 7-80** present curves representing the FS for each failure mode, plotted alongside the results from centrifugal experiments. In each figure, two vertical dashed lines indicate the range of seismic coefficients that trigger slope sliding. Most observed results just fall within this predicted range, except for the left slope in Case 3 (see **Fig. 7-76 (a)**) and the right slope of Case 4 (see **Fig. 7-77 (b)**), suggesting that the pseudo-static theory provides a satisfactory explanation for the dynamic stability of the benched slopes. In most prediction, mode ① has the most potential to happen,

which also aligns with the experimental results.

7.4.6 Variation of the slope state with ground motions

As mentioned in **Section 4.1.3**, the reduction ratio, r_d , expressed by **Eq. (4-45)** can characterize the slope state, ranging from the active state to a passive state. The model slopes experienced deformation under centrifugal loading during testing, resulting in varied states. For each slope, two r_d values can be calculated, yielding four r_d values intotal. For example, in the right slope of Case 1, a horizontal earth pressure sensor (P10, located along the centerline of the model) and two vertical earth pressure sensors (P6 on the back wall and P8 on the right wall) are located 9 cm above the bottom of the soil chamber, hence, by combining P10 with P6 and P8, two r_d values can be obtained (Tao & Pipatpongsa, 2023).

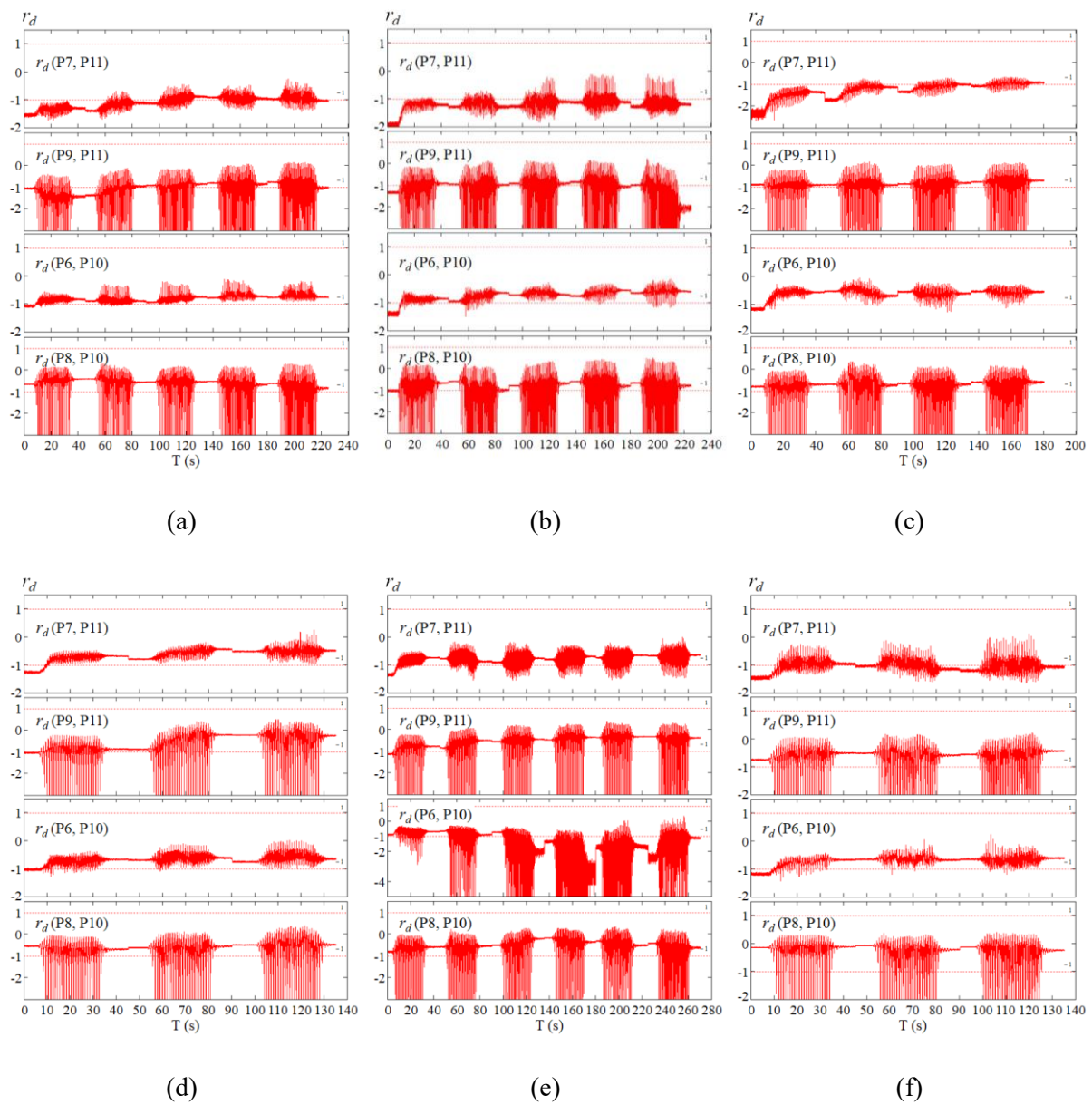


Fig. 7-81 Variation of reduction ratio with ground motions of the first group: (a) Case 1; (b) Case 3; (c) Case 4; (d) Case 5; (e) Case 6; (f) Case 7

The reduction ratio (r_d)'s variation with each shaking in the first group is shown in **Fig. 7-81**. When the model reached 50G, prior to any shaking, it had already achieved limit active state, with r_d around -1. The initial low-intensity shaking generally increased the r_d value, which could then decrease following stronger seismic loading. Moreover, r_d values calculated from pressure sensors on the side walls (P8 and P9) were more pronounced than those from sensors on the left and right walls (P6 and P7), indicating that the area near the slope surface experienced more significant state variation during shaking.

Fig. 7-82 illustrates the variation of reduction ratio with each shaking of the second group. Initial values of r_d measured by sensors on the side wall (P6 and P7) were lower than that with the sensors on the left and right walls, indicating the slope part near the slope surface experienced greater state variation before shaking. Additionally, r_d values calculated by pressure sensors on the side wall (P6 and P7) were less pronounced than from by sensors on the left and right walls (P8 and P9), suggesting that earth pressure fluctuates more significantly along the direction of shaking.

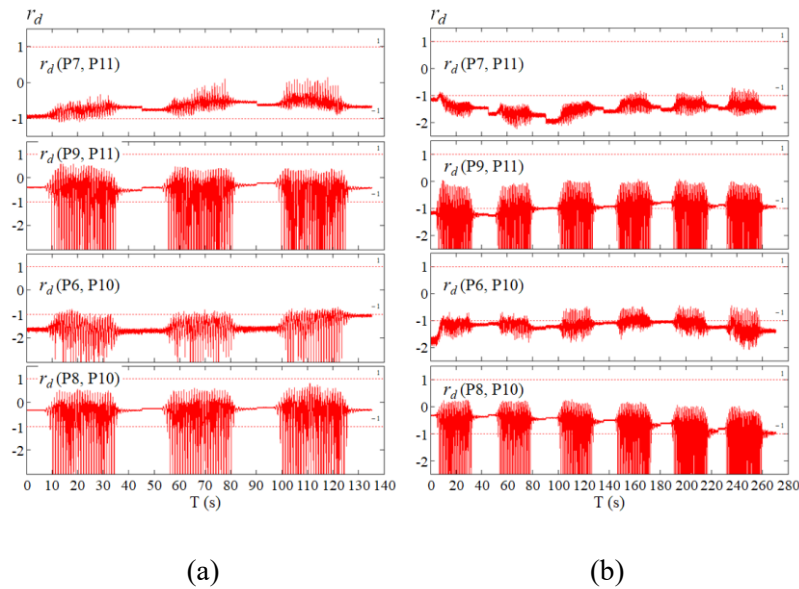
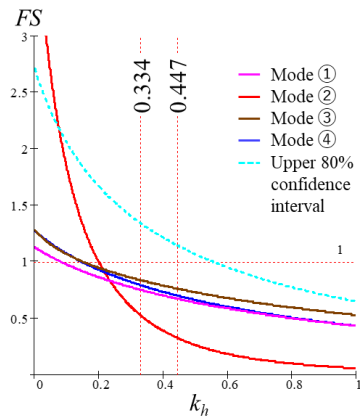


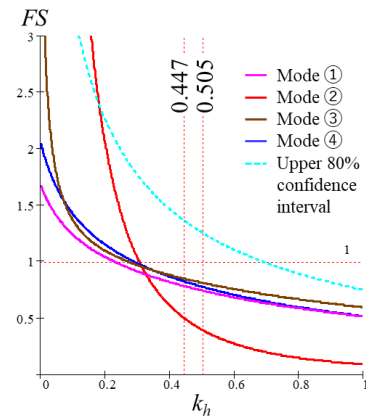
Fig. 7-82 Variation of reduction ratio with ground motions of the second group: (a) Case 1; (b) Case 2

7.4.7 Impact of lateral resistance considering 3D conditions

From the observations of r_d , the benched models reached a static state or even a passive state during the shaking; therefore, it is necessary to consider the lateral resistance. For failure modes ①, ③ and ④, r_d is set to 0, while for mode ②, it is assigned a value of 1, as the slope toe is pushed and likely reaches a limiting passive state (Stead & Eberhardt, 1997). The results are presented below.

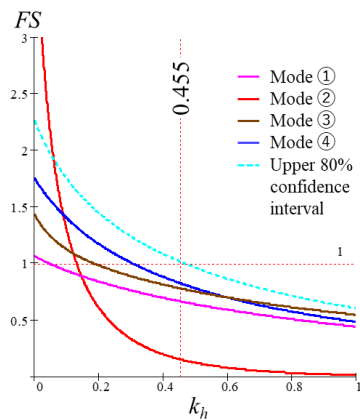


(a)

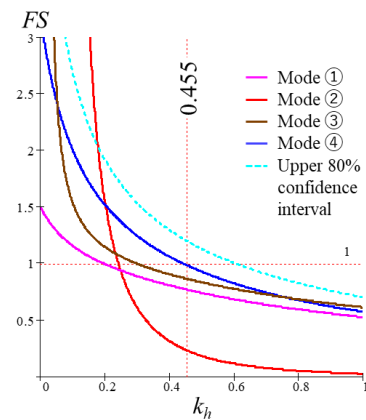


(b)

Fig. 7-83 Comparison between the prediction and the experiments of Case 1 of the first group considering the lateral resistance: (a) left slope; (b) right slope

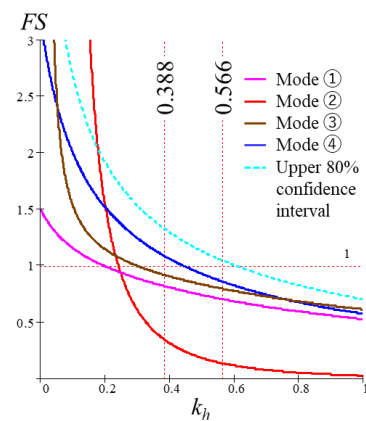
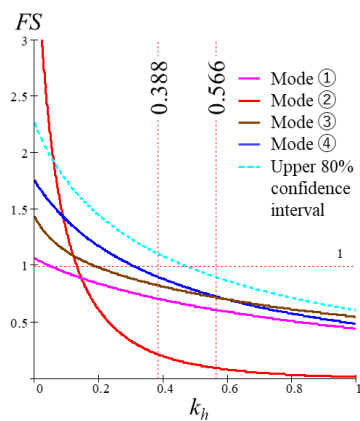


(a)



(b)

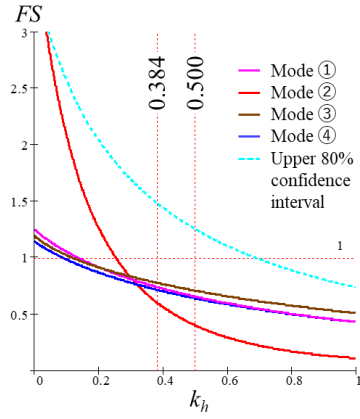
Fig. 7-84 Comparison between the prediction and the experiments of Case 2 of the first group considering the lateral resistance: (a) left slope; (b) right slope



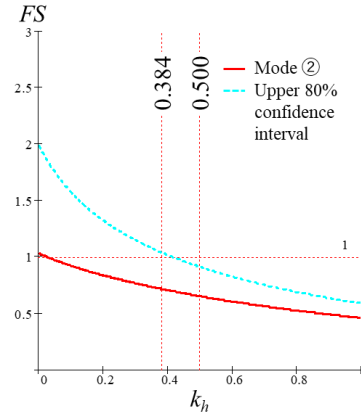
(a)

(b)

Fig. 7-85 Comparison between the prediction and the experiments of Case 3 of the first group considering the lateral resistance: (a) left slope; (b) right slope

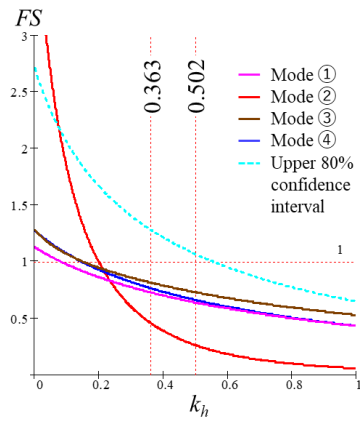


(a)

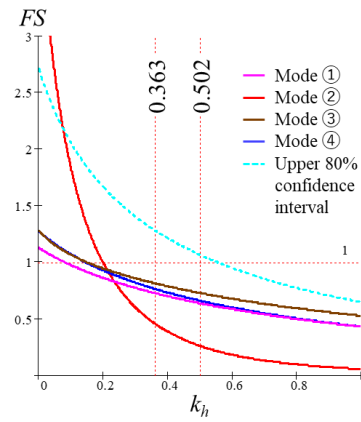


(b)

Fig. 7-86 Comparison between the prediction and the experiments of Case 4 of the first group considering the lateral resistance: (a) left slope; (b) right slope

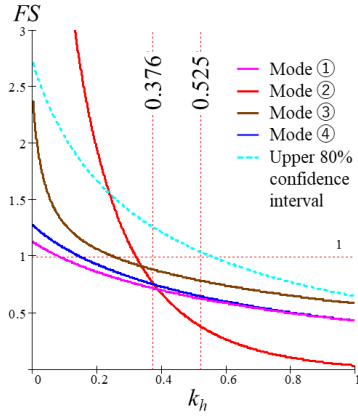


(a)

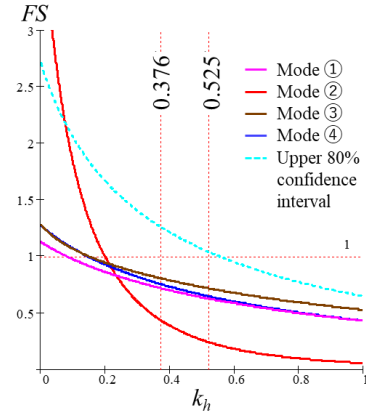


(b)

Fig. 7-87 Comparison between the prediction and the experiments of Case 5 of the first group considering the lateral resistance: (a) left slope; (b) right slope

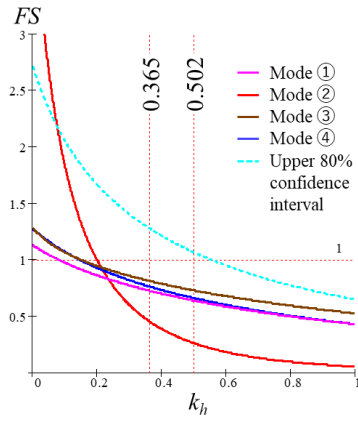


(a)

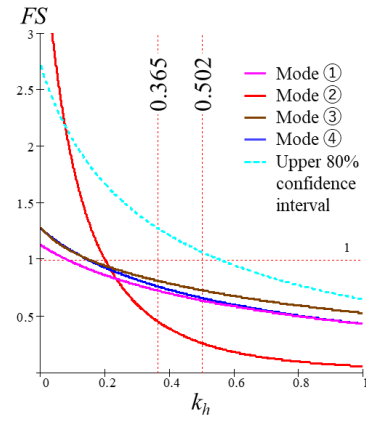


(b)

Fig. 7-88 Comparison between the prediction and the experiments of Case 6 of the first group considering the lateral resistance: (a) left slope; (b) right slope



(a)



(b)

Fig. 7-89 Comparison between the prediction and the experiments of Case 7 of the first group considering the lateral resistance: (a) left slope; (b) right slope

Fig. 7-83 to **Fig. 7-89** show an upward shift in FS curves for all failure modes. For the left slope in Case 3 (see **Fig. 7-76 (a)**) and the right slope of Case 4 (see **Fig. 7-77 (b)**), which are not accurately predicted under 2D conditions, the average experimental seismic coefficient falls within the predicted range when lateral resistance is considered. Thus, accounting for lateral resistance improves prediction accuracy.

7.5 Conclusions

This chapter examined the dynamic stability of benched slopes through dynamic centrifugal tests and pseudo-static methods. Various failure modes were observed and analyzed under both 2D and 3D conditions. Two different bench design approaches were applied to determine the optimal configuration for benched slopes. The key conclusions are as follows.

(1) Adding a first bench at the slope toe can enhance the slope's overall dynamic stability. Due to seismic amplification effects, the second bench is generally less stable than the first. Failure modes tend to shift from toe sliding in the original uniform slope to bench sliding on the second bench in the benched slope. Therefore, increasing the width of the first bench may not significantly improve overall stability; however, constructing a higher first bench, within certain limits, can provide greater stabilization.

(2) With equivalent excavation, increasing the number of steps in a benched slope can improve its seismic resistance. The failure of a benched slope under shaking was more controlled by the moment equilibrium rather than force equilibrium in some conditions.

(3) Geometric modifications through slope benching did not influence the interior seismic amplification factor, as this factor is an intrinsic characteristic of the soil.

(4) Since the slope can reach a static or even passive state during ground motions, lateral shear resistance cannot be overlooked; incorporating it can improve prediction accuracy for dynamic conditions.

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Chapter 8 Comparison of failure modes of benched slopes using centrifugal models and 3D DEM simulations

This chapter investigates the static and dynamic failure mechanism of benched slopes through simulations by the distinct element method (DEM). In centrifugal experiments, sliding modes of benched slopes are complicated and affected by various factors, such as model inhomogeneity caused by water redistribution, the compaction process, and the spatial nonuniformity of centrifugal loading. Additionally, observing the sliding surface is challenging due to the rapid sliding speed and the overlying soil that conceals the lower portion of the sliding surface. DEM simulations, however, can provide detailed particle-scale information, offering insights into the underlying principles of benched slope sliding.

In **Section 8.1**, the contact models used for sphere- sphere and sphere -wall contacts are introduced first. Microparameters required are calibrated by the results of direct shear tests.

According to the calibration and selected cases from centrifugal model tests, DEM slope models are constructed. The models' configurations are detailed in **Section 8.2**.

Section 8.3 first presents the simulation outcomes for uniform slopes under static and dynamic conditions. Subsequently, followed by the results for benched slopes under similar conditions.

The stress and displacement in the models are illustrated to analyze the influence of slope configuration and different loading methods. Additionally, **Section 8.4** discusses the evolving state of the force chain network during loading.

Lastly, the findings presented in this part are listed in **Section 8.5**.

8.1 Microparameters' calibration based on direct shear tests

To thoroughly elucidate the failure mechanism of benched slopes, the commercial software PFC3D (version 7.00) was employed. Before simulating the model slopes, the microparameters need to be calibrated. The Adhesive Rolling Resistance Linear Model (ARRLM) (Gilabert et al., 2007, 2008) (see **Fig. 8-1 (b)** and **(c)**), which is based on the Linear Model (LM) (see **Fig. 8-1 (a)**), was selected as the contact model to simulate the interaction behavior of sand particles. The LM can transmit normal and shear forces by connection springs, but it cannot transmit moment to resist relative rotation. For the ARRLM, the rolling resistance is taken into consideration, by which the internal moment is incremented linearly with the accumulated relative rotation of the contacting pieces at the contact point (Gilabert et al., 2007). Additionally, a short-range attraction, which is a linear approximation of the van der Waals force law, is used to represent the cohesion in cohesive granular materials.

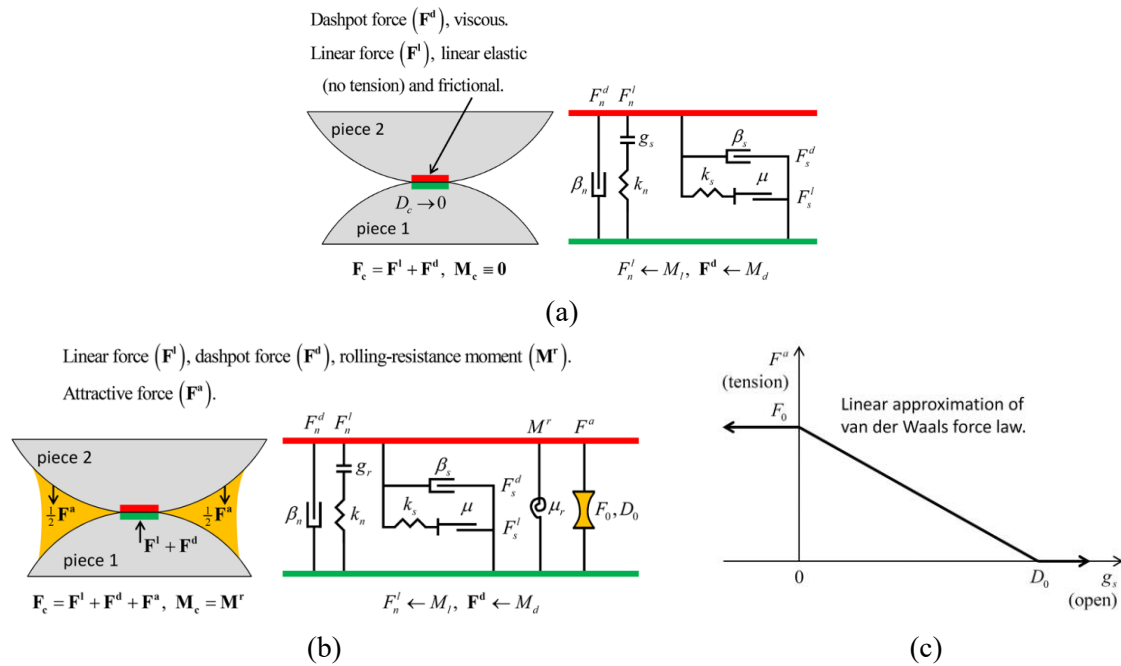


Fig. 8-1 Adhesive rolling resistance linear model: (a) Behavior and rheological components of the linear model; (b) behavior and rheological components of the adhesive rolling resistance linear model; (c) attractive force versus surface gap for the adhesive rolling resistance linear model (Gilabert et al., 2007)

A trial-and-error procedure (Jin et al., 2024) was conducted by adjusting the input microparameters in a series of numerical direct shear tests (NDST), to obtain the parameters that match the macro-mechanical response observed in experimental direct shear tests (EDST), which was introduced in **Chapter 3**. According to the ARRLM, six microscopic parameters need to be determined. These parameters are effective modulus (E^*), normal-to-shear stiffness ratio (κ^*), friction coefficient (μ), rolling friction coefficient (μ_r), maximum attractive force (F_0), and attraction range (D_0), but not all micro-properties can be calibrated (Sadek et al., 2011), some must be preselected before the calibration

of others. In this study, for all the simulated particles, μ was arbitrarily set to a common value of 0.5, and κ^* was set to 1.5 for simplicity (CUI et al., 2022; Sadek et al., 2011). Therefore, the parameters left to be determined are E^* , μ_r , F_0 and D_0 . For the particle-wall contact, the Linear Model was used with a high E^* of 1 GPa and κ^* of 1.5. The wall in the NDST was assumed to be completely smooth, with μ equal to 0.

For the NDST, a cuboid shear box was modelled, comprising twelve flat walls (see **Fig. 8-2**). Five walls were used to construct the upper part of the shear box, another five walls formed the lower part, and two additional walls were positioned at the shear surface level to prevent particles from escaping during the shear process. For the EDST, the shear box was a cylinder, having a diameter of 0.06 m and a height of 0.02 m. These dimensions were converted to the width and height of the cubic container by multiplying a coefficient of 0.22 as shown in **Fig. 8-2 (a)**. This coefficient was chosen to reduce the box size, thereby keeping the number of particles within a reasonable range, given the constraints of computational capacity.

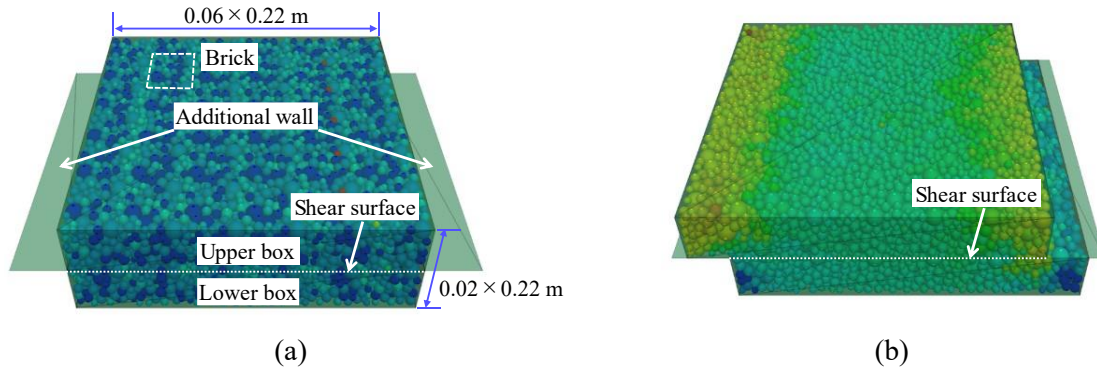


Fig. 8-2 Shear box with model particles in NDST: (a) before shearing; (b) after shearing

PFC3D can create a particle assembly based on the designated grain size distribution curve and porosity using the 'particle distribute' command. To enhance the homogeneity of the specimen, a brick was created first (see **Fig. 8-3**). The brick is derived by creating an assembly of particles within periodic space and then storing it in compact form. Copies of this assembly can then be fitted together perfectly, because the geometrical arrangement of particles on one side of a brick is the negative image of that on the opposite side (Group., 2024). Then, this brick was replicated multiple times to construct the specimen (**Fig. 8-2 (a)**).

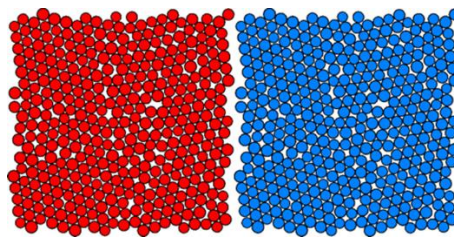


Fig. 8-3 Diagram of the brick

Fig. 8-4 shows a comparison of the grain size distribution between the real material and the DEM model. Particles with diameters less than 0.25 mm were excluded from the simulation, and their volume percentage was incorporated into the 0.25 mm to 0.425 mm range to reduce computational costs. The median particle diameter, d_{50} , of the simulation was 0.39 mm. It was slightly larger than the d_{50} of 0.33 mm for the real sand. The ratio of the numerical shear box height, which was the smallest characteristic model length, to the median diameter of the DEM particle (L/d_{50}) was 11. This value was sufficiently large to avoid the boundary effect (Zhang et al., 2018). The shear displacement rate for EDST was 0.2 mm/min, which was increased by a factor of 1000 for the NDST. To address the inertia effects caused by the loading rate, a relatively high damping ratio of 0.7 was adopted for the NDST.

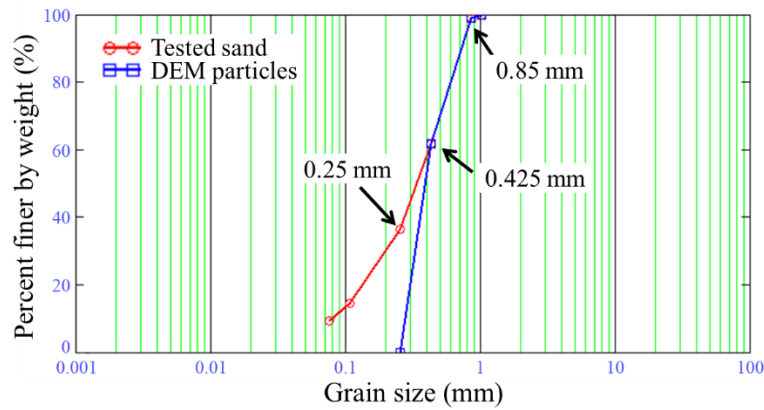


Fig. 8-4 Comparison of grain size distribution curves between real sand and DEM particles

Initially, the simulation was conducted for the dry Hiroshima sand with a compaction degree (D_c) of 80%. The DEM particles were assigned the same particle density as the real Hiroshima sand (2633 kg/m³). The sample's porosity was obtained using the particle density and the total density. 13,284 particles were created for the specimen. At first, μ_r , F_0 and D_0 were set to zero, and E^* was predetermined to a common value of 100 MPa. Values of μ_r and E^* were adjusted iteratively to align the virtual stress-strain curves with the real ones. Ultimately, $\mu_r=0.16$ and $E^*=300$ MPa were determined. During the consolidation stage of the NDST, the upper box was fixed, and normal pressure was applied to the lower box's bottom wall, consistent with the procedure used in the EDST. During shearing, this pressure was maintained constant using a servo command, and the upper box moved to the left to shear the specimen. **Fig. 8-5 (a)** presents the stress-strain curves from both EDST and NDST for dry sand samples, after calibration, under different confining pressures of 40, 60, 80, 100, and 120 kPa.

Due to matric suction, the humid Hiroshima sand prepared for the model exhibits higher strength compared to the dry sand. Here, the F_0 and D_0 were calibrated to represent the additional cohesion caused by the suction. Those two parameters were set to non-zero values only for contacts established before shearing; any newly created contacts during shearing were assigned a value of zero. Since the water content decreased in most parts of the slope, EDST with $D_c=80\%$ and $w=6\%$ were used for

calibrating the humid sand. The results are $F_0=1.2\times10^{-4}$ N and $D_0=0$. **Fig. 8-5 (b)** shows the stress-strain relationships from both EDST and NDST for humid sand samples after calibration. Finally, the material properties obtained from the calibration are listed in **Table 8-1**.

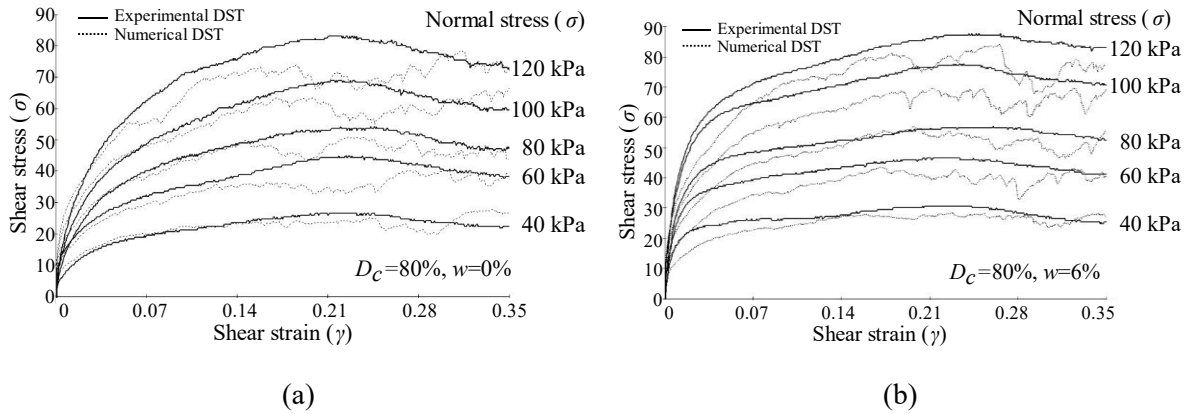


Fig. 8-5 Shear stress – shear strain curves under different normal pressures for Hiroshima sand ($D_c=80\%$): (a) dry sand (b) humid sand

Table 8-1 Material properties of DEM analysis to simulate Hiroshima sand ($D_c=80\%$)

	Material property	Value (units)
Particle	Density	2633 kg/m ³
	Damping ratio	0.7
Particle-particle	Effective modulus (E^*)	300 MPa
	Normal-to-shear stiffness ratio (κ^*)	1.5
	Friction coefficient (μ)	0.5
	rolling friction coefficient (μ_r)	0.16
	Maximum attractive force (F_0)	1.2×10^{-4} N
	Attraction range (D_0)	0 m
Particle-wall	Effective modulus (E^*)	1 GPa
	Normal-to-shear stiffness ratio (κ^*)	1.5
	Friction coefficient (μ)	0

In the same way, the parameters for Hiroshima sand with D_c of 90% are calibrated. The stress – strain curves of the DST and the calibration are presented in **Fig. 8-6** and **Table 8-2**, respectively.

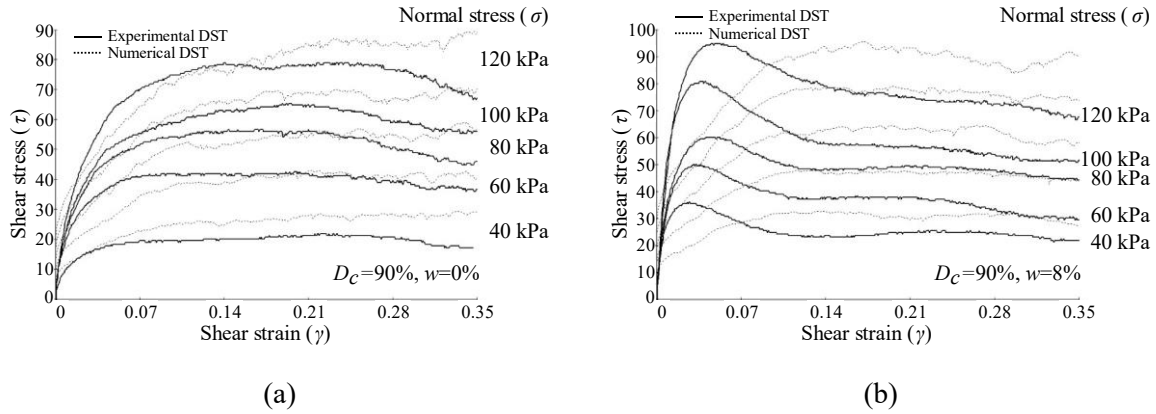


Fig. 8-6 Shear stress – shear strain curves under different normal pressures for Hiroshima sand ($D_c=90\%$): (a) dry sand (b) damp sand

Table 8-2 Shear stress – shear strain curves under different normal pressures for Hiroshima sand ($D_c=90\%$): (a) dry sand (b) humid sand

	Material property	Value (units)
Particle	Density	2633 kg/m ³
	Damping ratio	0.7
Particle-particle	Effective modulus (E^*)	300 MPa
	Normal-to-shear stiffness ratio (κ^*)	1.5
	Friction coefficient (μ)	0.5
	rolling friction coefficient (μ_r)	0.2
	Maximum attractive force (F_0)	2×10^{-4} N
	Attraction range (D_0)	0 m
Particle-wall	Effective modulus (E^*)	1 GPa
	Normal-to-shear stiffness ratio (κ^*)	1.5
	Friction coefficient (μ)	0

8.2 Setup of DEM slope models

8.2.1 Model generation

Following the sample generation method used in the NDST, a cuboid sample was generated to represent the foundation of the centrifugal model, as depicted in previous chapters. Its dimensions were determined by taking half the length of the actual foundation, as it was intended to simulate only one model slope. Then, all dimensions were multiplied by a size reduction factor of 0.063 to reduce the total number of particles. Finally, 47,070 particles were generated for static cases with a D_c of 80% and 52,380 particles for dynamic cases with a D_c of 90%.

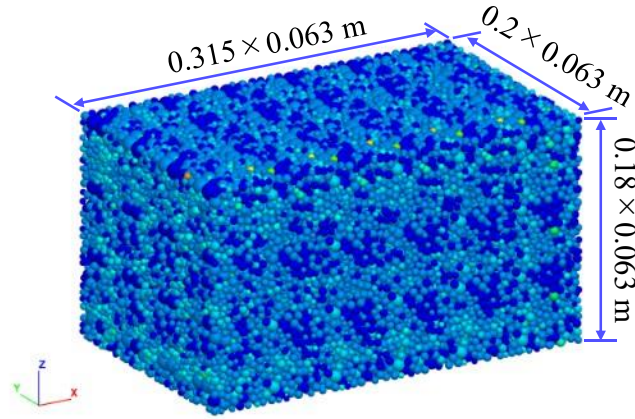


Fig. 8-7 Dimensions of the foundation in DEM

For the foundation sample, $L/d_{50}=32$. While most parameters in the simulation of the benched slopes remained consistent with those in the NDST, the wall-particle friction (μ) was set to 0.2 for static cases and 0.4 for dynamic cases to account for the lateral friction provided by the wall of the soil chamber. The interface frictional angle between the material and the aluminum were measured by Wongchana (2024).

The parameter of attractive force, F_0 , must be scaled to simulate slope models accurately using the principles of centrifugal modeling. According to Weng et al. (2016) and Weng et al. (2017), the force term is scaled with a ratio of $1/N^2$, between the model scale and the prototype scale, where N represents the scaling factor for the reduced model. In this study, $N=1/0.063$. However, unlike their model construction method, which simultaneously scaled up the sizes of both the model and particles, this simulation reduced only model dimensions, leaving particle dimensions unchanged. Consequently, the number of particles was scaled down by around $1/N^3$, which implies that the total attractive force was also reduced by $1/N^3$ approximately. To compensate for this reduction, the attractive force was increased by the factor of N here. Additionally, Weng et al. suggested a modified parameter, a of 0.3 to adjust the deformation; however, the simulation results in this study were satisfactory without this adjustment.

8.2.2 Simulation procedure

In the centrifugal modeling, the foundation was initially cut to shape slope surfaces, after which the model slope was subjected to increasing centrifugal loading. The entire loading process could take approximately 1500 s for the static case. Simulating the exact loading process can be time consuming and impractical in DEM.

To address this issue, the foundation in the simulation was first loaded by an increased gravitational force to reach an equilibrium state. Then, part of particles was deleted to create the benched slope. Subsequently, the benched slope was loaded to deform under the increased gravitational force, which mimicked the centrifugal loading in the model test. In the centrifugal model, the centrifugal loading is equivalent to the gravitation multiplied by a scaling factor (n). In the simulation, the scaling factor was

further increased by dividing it with the size reduction factor (0.063) to achieve a stress state similar in centrifugal model (i.e., the simulated slope and the centrifugal slope model have the same prototype size).

In the simulation, for the static cases, the n varies for the foundation construction stage, while the n for dynamic cases was set to 50 constantly, which is identical to that adopted in the centrifugal modeling. The equivalent input wave the from the centrifugal modeling according to the scaling principle was then applied to the dynamic simulation to trigger slope sliding.

8.3 Simulation results

8.3.1 Simulation results of benched slopes under static conditions

8.3.1.1 Uniform slopes

The centrifugal model test of Case 2 in chapter 4 was selected as the basis of the DEM simulation of uniform slopes under static conditions. This case had two slopes in the narrow soil chamber, including a left slope with an inclination of 55° and a right slope of 65° .

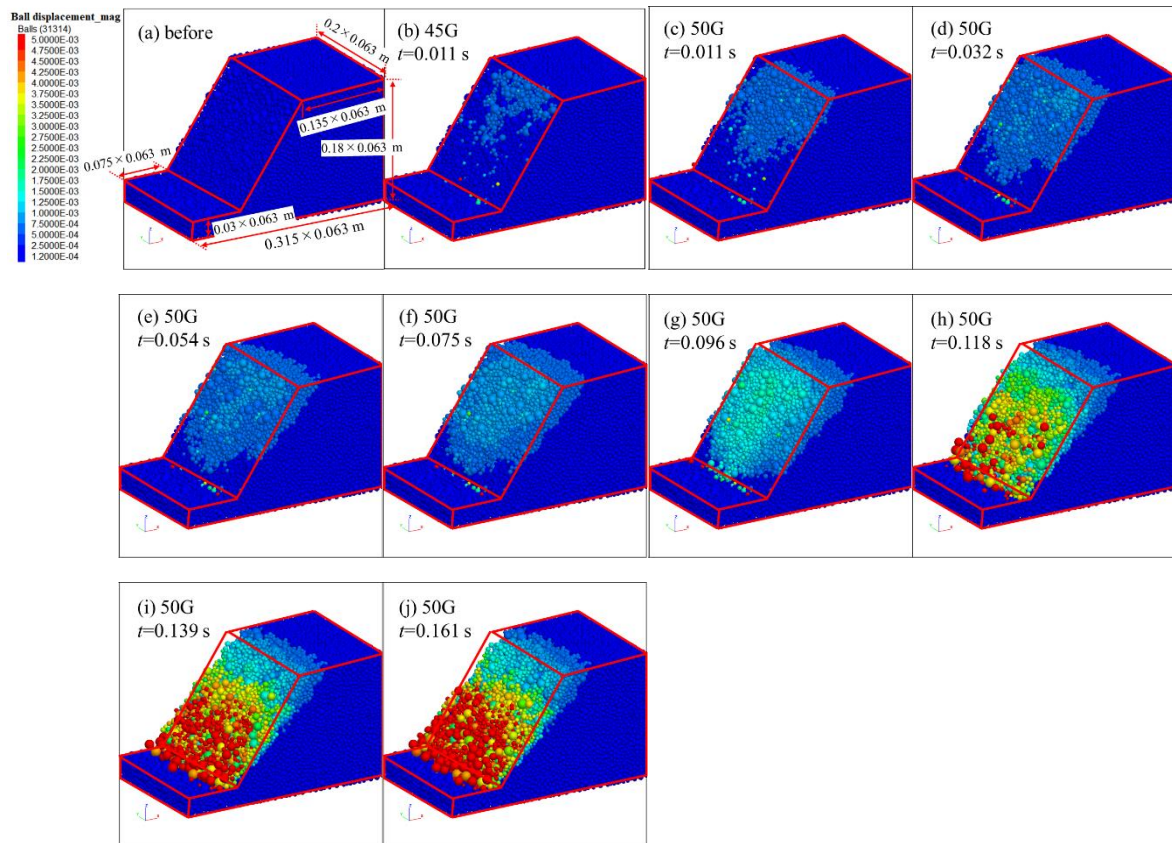


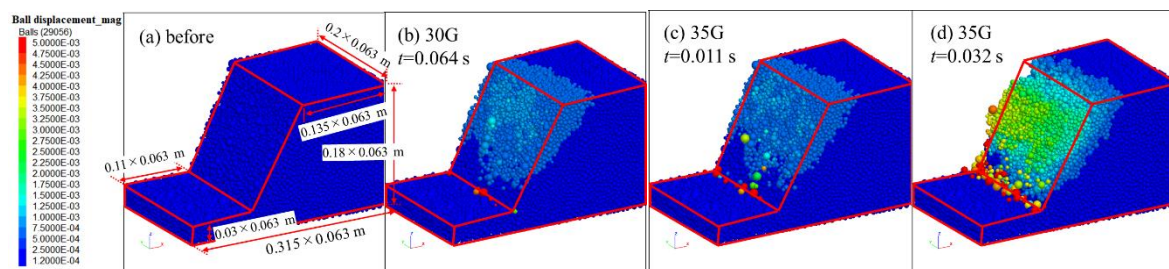
Fig. 8-8 Sliding process of uniform slopes with a slope angle of 55° simulated by DEM

The left slope was simulated first, as presented in **Fig. 8-8**. The **Fig. 8-8 (a)** shows the state of the slope immediately after removing the undesirable particles (i.e., at the slope formation). The slope was modeled with 31,314 particles in total. The accumulated displacement during both the specimen

generation and loading processes was reset to zero to facilitate the observation of the subsequent sliding process. The maximum and minimum display boundary of the displacement were set to 5×10^{-3} m and 1.2×10^{-4} m, respectively, which cover the displacement range of most particles. The outline of the undeformed slope was indicated by red lines. The centrifugal loading shown in each subgraph is the equivalent value in the centrifugal modeling. Yang et al. (2014), and Chiu and Weng (2019) noted that the friction coefficient between the sliding block and the sliding surface decreased as the velocity increased. Therefore, μ_r was set to zero for all new contacts between particles established during the sliding process.

When a loading of 45G was applied, deformation began at the slope crest (see **Fig. 8-8 (b)**). Although a few particles rolled down, no distinct sliding surface was formed, and the slope stabilized after 0.011 s. Next, the loading on the foundation was increased to 50G, and the sliding process was simulated once more. The displacement figure was selected with an interval of 2×10^5 timesteps. After part of particles were removed to shape the slope face, deformation began at the crest (see **Fig. 8-8 (c)**), the deformed area develop from the slope crest to the crest toe (see **Fig. 8-8 (d)**), followed by sliding in the front part of the deformed area (see **Fig. 8-8 (f)**). Eventually, the particles slid along the sliding surface and accumulated on the toe (see **Fig. 8-8 (j)**), and the model nearly reached equilibrium. The state of the slope remained largely unchanged afterward, so the subsequent graphs are not presented.

The simulated centrifugal loading required to fail the simulated slope was 50G, which deviates from the 67G observed for the left slope in Case 2 of the first group, **Chapter 4**. It may be caused by reduced water content from the initial value of 8% on the slope part as shown in **Table 4-4**. This distribution of water content resulted from evaporation and gravitational forces during the sample preparation stage, as well as from centrifugal loading during the test. This variation within the model led to differences in shear strength at various slope heights. Another possible reason for this phenomenon is that the centrifugal loading was not uniform across the model height. The upper part of the model, being closer to the centrifuge pivot, experienced less loading. Additionally, in the simulation, the sliding surface extended across the entire model breadth, whereas only a portion of the slope failed during the first sliding in the centrifugal model. This difference is attributed to that the centrifugal model was compacted by hand, leading to inevitable inhomogeneity. The area of the model near the wall was less compacted than the central part.



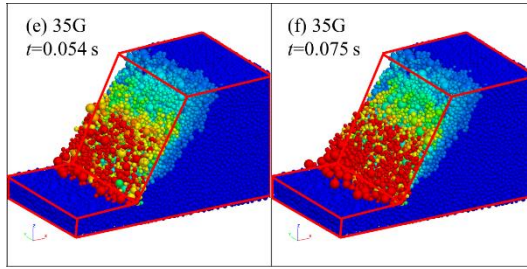


Fig. 8-9 Sliding process of uniform slopes with a slope angle of 65° simulated by DEM

The right slope was then simulated, as shown in **Fig. 8-9**. It had 29056 particles. Similarly, this uniform slope with a high inclination can remain stable under 30G. While a loading of 35G was applied, the sliding happened. In the centrifugal tests, for the right slope in Case 2 of the first group, **Chapter 4**, the failure occurred at 32G, which is similar to the centrifugal results.

8.3.1.2 Benched slopes

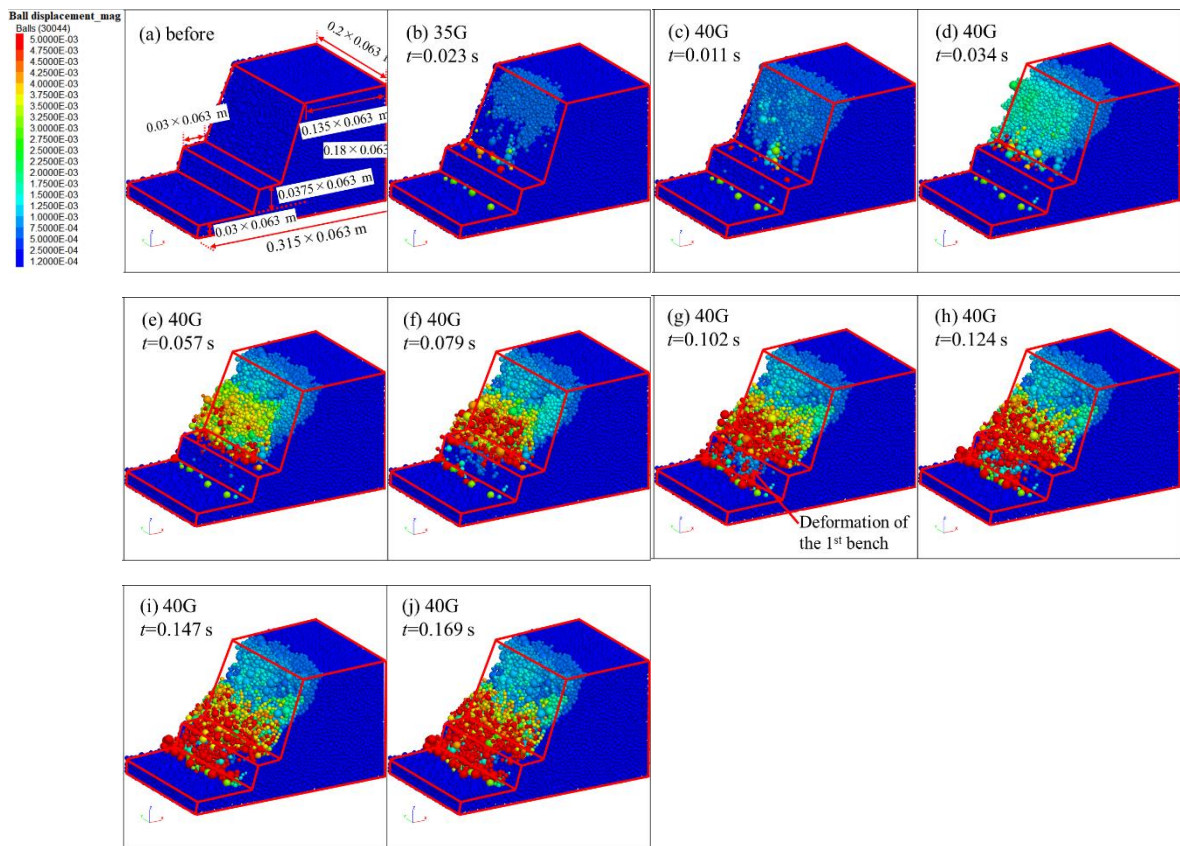


Fig. 8-10 Sliding process of benched slopes with the lower bench simulated by DEM

Case 4 in **Chapter 5** was selected for the simulation of benched slopes under static loading. It had two benched slopes in the narrow soil chamber. The left and right first benches were 7.5 cm and 3.75 cm in height, respectively, in model scale. The first and second benches on the left slope were equal in

height, while the right slope's first bench was half as tall as the left one. Apart from the height of the first bench, all other geometrical parameters of both slopes were identical. The first bench width was 3 cm, the total slope height was 15 cm, the overall model height was 18 cm, the model's breadth was 20 cm, and the distance between the soil chamber wall and the crest was 13.5 cm.

The right slope with a lower first bench was simulated first, as shown in **Fig. 8-8**. It had 30044 particles. When a loading reached 35G, deformation began at the crest (see **Fig. 8-10 (b)**). Although a few particles rolled down, no distinct sliding surface was formed, and the slope stabilized after 0.023 s. Next, the loading on the foundation was increased to 40G. After part of particles were removed to shape the slope face, deformation began at the crest (see **Fig. 8-10 (c)**), followed by sliding in the front part of the deformed area (see **Fig. 8-10 (d)**). This sliding was confined to the second bench until part of the first bench was destroyed by the sliding body in 0.102 s (see **Fig. 8-10 (g)**). Eventually, the sliding body covered the first bench, and the model nearly reached equilibrium (see **Fig. 8-10 (j)**). The state of the slope remained largely unchanged afterward, so the subsequent graphs are not presented.

The simulated centrifugal loading required to fail the simulated slope was 40G, which deviates from the 58G observed in the centrifugal model. It may be caused by reduced centrifugal loading of the second bench in the centrifugal model.

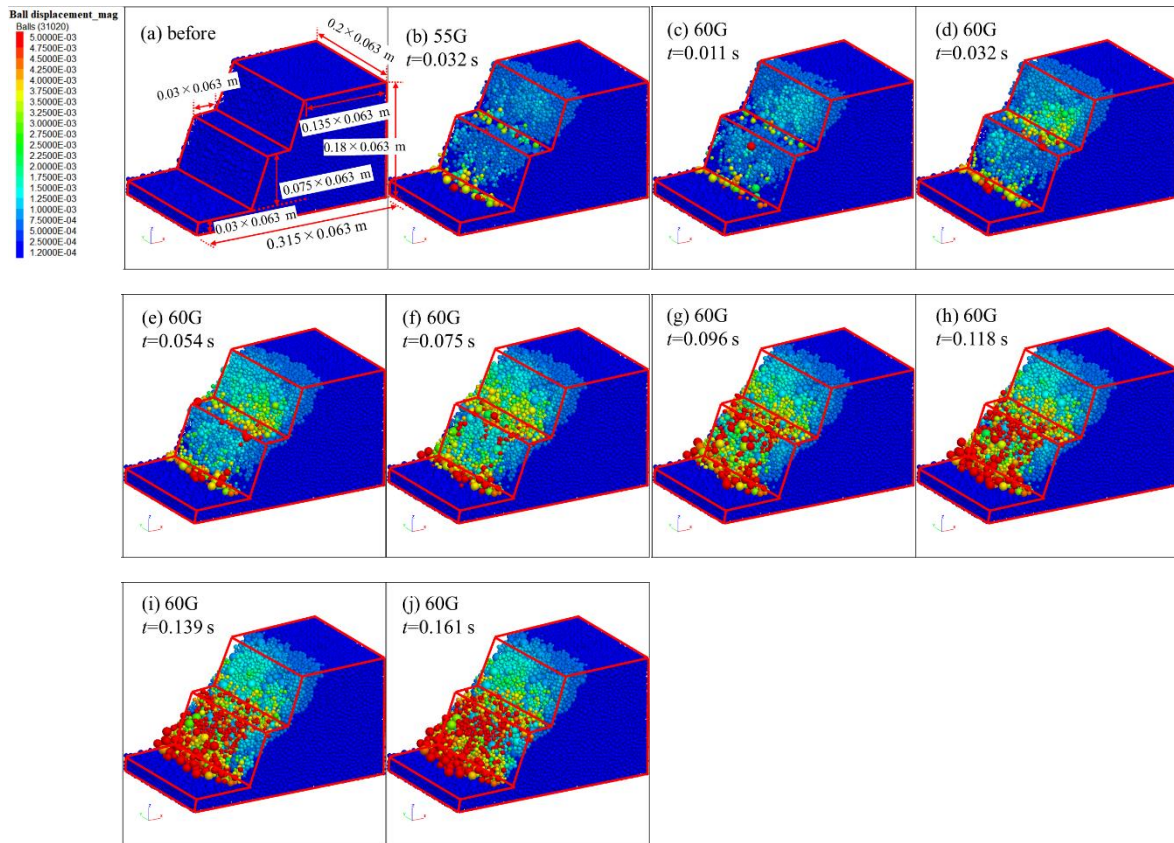


Fig. 8-11 Sliding process of benched slopes with the higher bench simulated by DEM

The left benched slope with the higher bench was then simulated, as presented in **Fig. 8-11. Fig. 8-11**

(a) shows the state of slope immediately after the formation of the benched slope. Similar to the simulation of the benched slope with a lower bench, the accumulated displacement was reinitialized, and the same displacement range was adopted. A total of 31,020 particles were simulated for this benched slope.

When a loading of 55G was applied to the model, deformation began simultaneously in the front areas of the first and second benches (see **Fig. 8-11 (b)**). Within 0.032 s, the slope stabilized with increased deformation and some particles rolled down. Subsequently, the loading was increased to 60G. The slope exhibited more deformation in **Fig. 8-11 (d)** compared to **Fig. 8-11 (b)**, even though the same duration of 0.032 s was taken. Deformation continued to develop until 0.075 s, at which point sliding began simultaneously on the first and second benches (see **Fig. 8-11 (f)**). At last, the sliding body accumulated at the toe, and the model approached a near equilibrium state (see **Fig. 8-11 (j)**). The state of the slope remained largely unchanged afterward, so the subsequent graphs are not included.

The simulated centrifugal loading required to fail the slope was 60G, which is nearly identical to the 59G recorded in the centrifugal model. However, the failure pattern observed in the simulation differed from that in the centrifugal modeling. In the simulation, both the first and second benches failed simultaneously, whereas the second bench remained stable in the centrifugal test. The slope in the DEM simulation was more homogeneous compared to the real slope. Additionally, the simulated centrifugal loading was uniform across all particles, whereas in the centrifugal test, the loading varied with height.

8.3.2 Simulation results of benched slopes under dynamic conditions

8.3.2.1 Uniform slopes

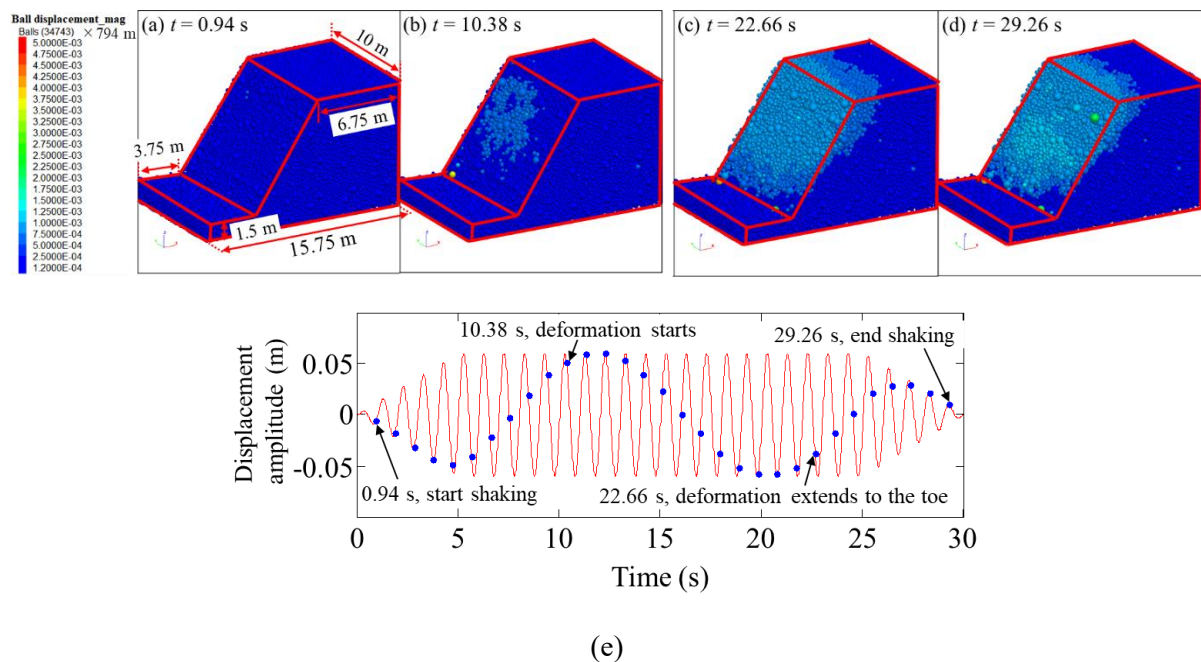


Fig. 8-12 Sliding process of a uniform slope with an inclination of 55° subjected to the ground motion with a magnification of 1.2 (prototype scale)

A uniform slope with an inclination of 55° was simulated using the material properties obtained in **Table 8-2**. The model included 34743 particles. The amplitude of the input wave was varied several times to determine the sliding-inducing magnitude. It was found that the slope could remain stable when the magnification number was 1.2 and failed at a magnification of 1.5. During each simulation process, the displacement of the model was recorded every 10^4 timesteps. The results are presented as follows in prototype scale (50G).

In **Fig. 8-12**, the dots represent the recorded time intervals during the shaking. After the commence of shaking, the deformation of slope started at 10.38 s, when the slope moves backwards around the peak displacement. The deformation range continued to expand with the shaking. It extended to the slope toe at 22.66 s. The slope did not slide at the end of shaking.

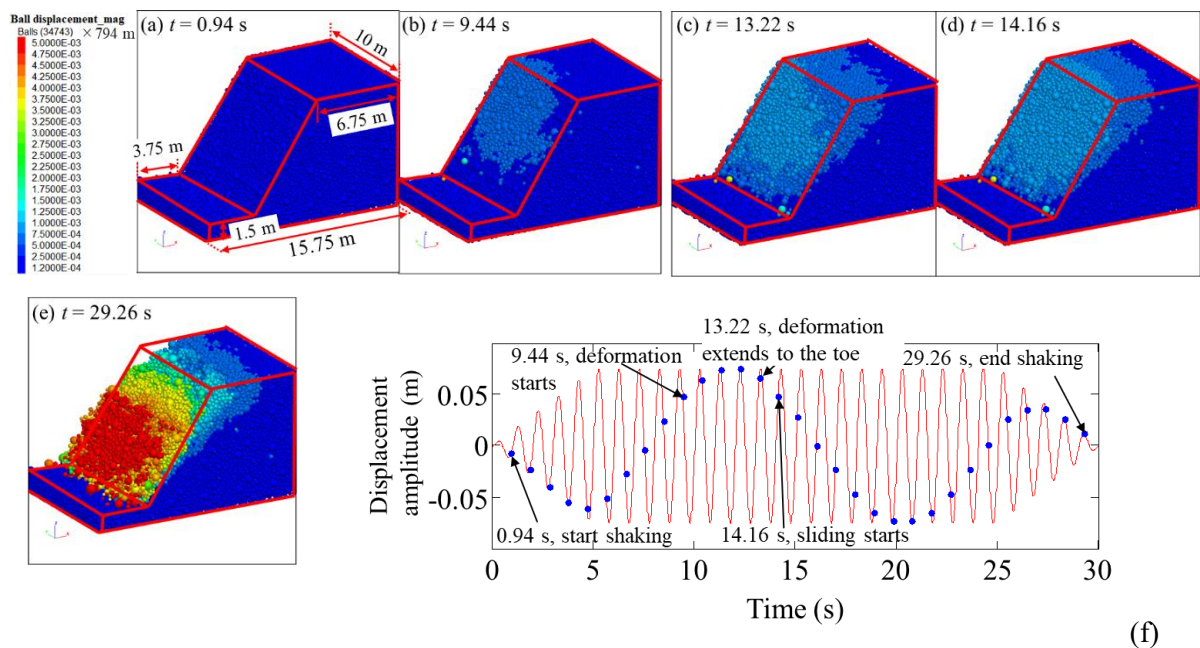


Fig. 8-13 Sliding process of a uniform slope with an inclination of 55° subjected to the ground motion with a magnification of 1.5 (prototype scale)

With a larger intensity of shaking having a magnification of 1.5, the slope deformation became more obvious in **Fig. 8-13**. The deformation started around the same time as the previous one, at 9.44 s. The deformation developed much quicker, extending to the slope toe at 13.22 s. Right after it, the sliding happened at 14.16 s. At the end of the shaking, the sliding body accumulated at the toe.

In the centrifugal test, the slope with the same configuration did not fail until a magnification of 1.9, which exhibited a higher strength than the DEM model.

8.3.2.2 Benched slopes

With the same inclination as the uniform slope, a benched slope with a high bench, 3 m high in prototype scale, was simulated, having 36542 particles. It was found that the slope could remain stable when the

magnification was 1.9 and failed at a magnification of 2.5, which was similar to the centrifugal test.

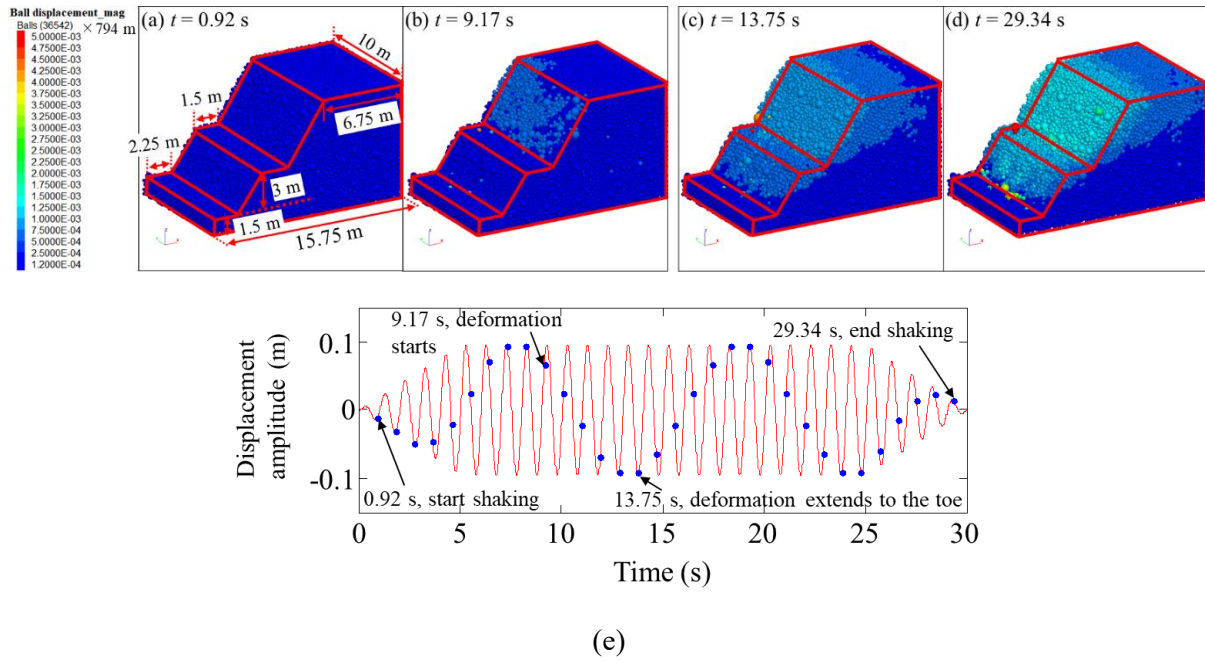


Fig. 8-14 Sliding process of a benched slope with an inclination of 55° subjected to the ground motion with a magnification of 1.9 (prototype scale)

As shown in Fig. 8-14, the deformation began at 9.17 s when the magnification was 1.9. It started on the second bench. The deformation extended to the slope toe at 13.75 s. At the end of the shaking, the slope did not fail.

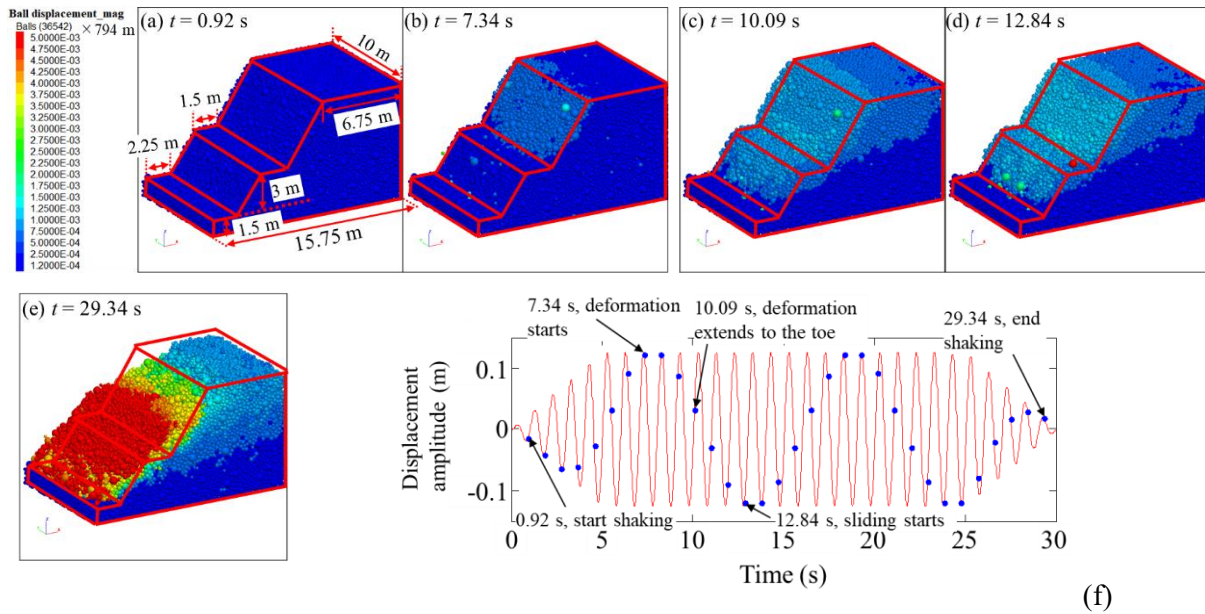


Fig. 8-15 Sliding process of a benched slope with an inclination of 55° subjected to the ground motion with a magnification of 2.5 (prototype scale)

The magnification of the shaking was increased to 2.5, as shown in Fig. 8-15. The deformation started at an earlier time of 7.34 s. The deformation extended to the slope toe at 10.09 s, followed by the sliding commencing at 12.84 s. At the end of the shaking, a toe sliding finished, with the sliding body accumulated at the toe. In the centrifugal test, only the second bench failed most of the time.

From the simulation results of the uniform slopes and benched slopes, the onset of deformation occurred earlier as the amplitude of the input wave increased. The deformation always started at the crest. With a bench part, the slope model can resist seismic excitation with a higher intensity.

8.4 Discussion

8.4.1 Deformation patterns

8.4.1.1 Deformation patterns affected by the slope configurations under static conditions

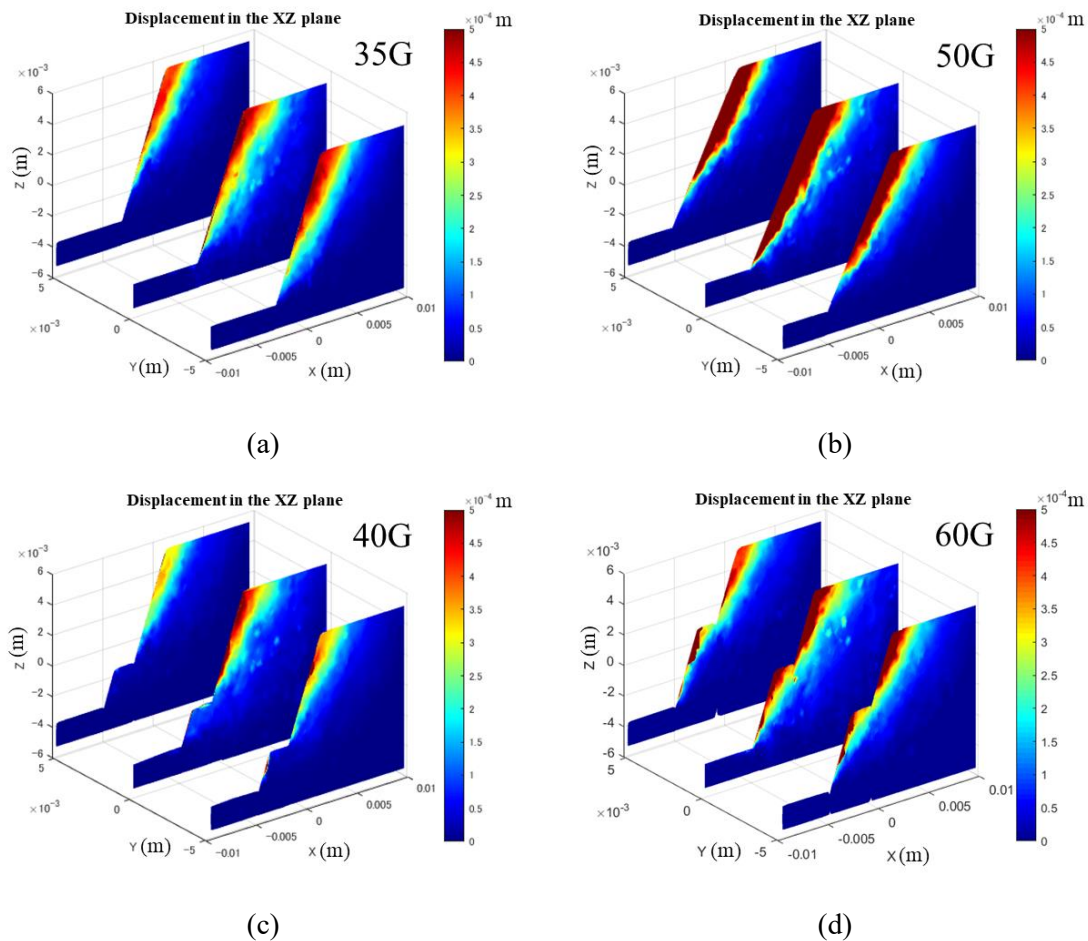


Fig. 8-16 Contour slices for displacement distribution in slopes with varying configurations before sliding: (a) uniform slope (65°) at 35G; (b) uniform slope (55°) at 50G; (c) benched slope (55°) with a low first bench at 40G; (d) benched slope (55°) with a high first bench at 60G

With varied slope configurations, slopes' failure modes under static conditions can be changed. It was shown by previous chapters by centrifugal experiments and theoretical explanations. The DEM

simulations in **Section 8.3.1** verified the trend of mode transition. In this section, more details will be presented.

The displacement distributions in slopes with varying configurations, just before the sliding happen, are illustrated in **Fig. 8-16** by three contour slices near the slope boundaries and along the centerline. The accumulated deformation before failure of the uniform slope of 65° (see **Fig. 8-16 (a)**) was less than that of the uniform slope of 55° (see **Fig. 8-16 (b)**). It was because that the slope with a higher inclination was less stable and failed at a relatively low loading condition. Similarly, the deformation of the benched slope with a low first bench (see **Fig. 8-16 (c)**) was less than that of the benched slope with a high first bench (see **Fig. 8-16 (d)**). This indicated that the bench can restrict the slope deformation.

Although the benched slope with a low bench was subjected to a higher load than the steeply inclined slope, and the benched slope with a high bench experienced a higher load than the less inclined slope, the deformations of the benched slopes were smaller than those of their respective counterparts. Additionally, for slopes with the same inclination, increasing the bench height raised the failure load from 35G to 40G, and then to 60G. This implied that the bench can stabilize the slope and increase its capability to withstand gravitational loading.

For all slope models, the deformation of the center slice was larger than the boundary slices. It meant that the sliding surface of the models with lateral confinements was actually a 3D curved surface.

The toe sliding was dominated for the uniform slope, while the bench sliding became more common for the benched slope. When the first bench is low, the slope's upper part was prone to failure. Having a higher bench, the possibility of sliding on the first and second bench became closer.

8.4.1.2 Deformation patterns affected by the slope configurations subjected to ground motions

Because the automatically calculated timesteps differed slightly between the uniform model and the benched model, two types of slopes experienced different moments of acceleration or deceleration in the corresponding sampling series (see **Fig. 8-17**). Thus, the 16.50 s, when the uniform slopes returned to the original location and 15.59 s, when the benched slope was near the cross point, are selected for comparison.

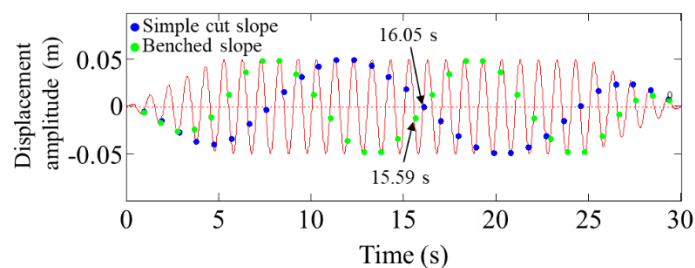


Fig. 8-17 Sampling moments for two types of slopes

The deformation response of slopes with different geometrical configurations at the same time point during the shaking was shown in **Fig. 8-18**. At this point, all slopes returned to their original location. Thus, no translational displacement existed. For the uniform slope, a larger deformation happened with a higher amplitude of the input wave. The potential failure surface was parallel to the slope face. For the benched slope, the accumulated displacement subjected to the input wave with higher amplitude was larger. When the magnification was 1.9, the deformation at the second bench was notably larger. With a higher magnification of 2.5, this difference became less obvious. The deformation level and area at 2.5 was larger than that at 1.9. The potential failure surface for the benched slope was always a 3D curved surface.

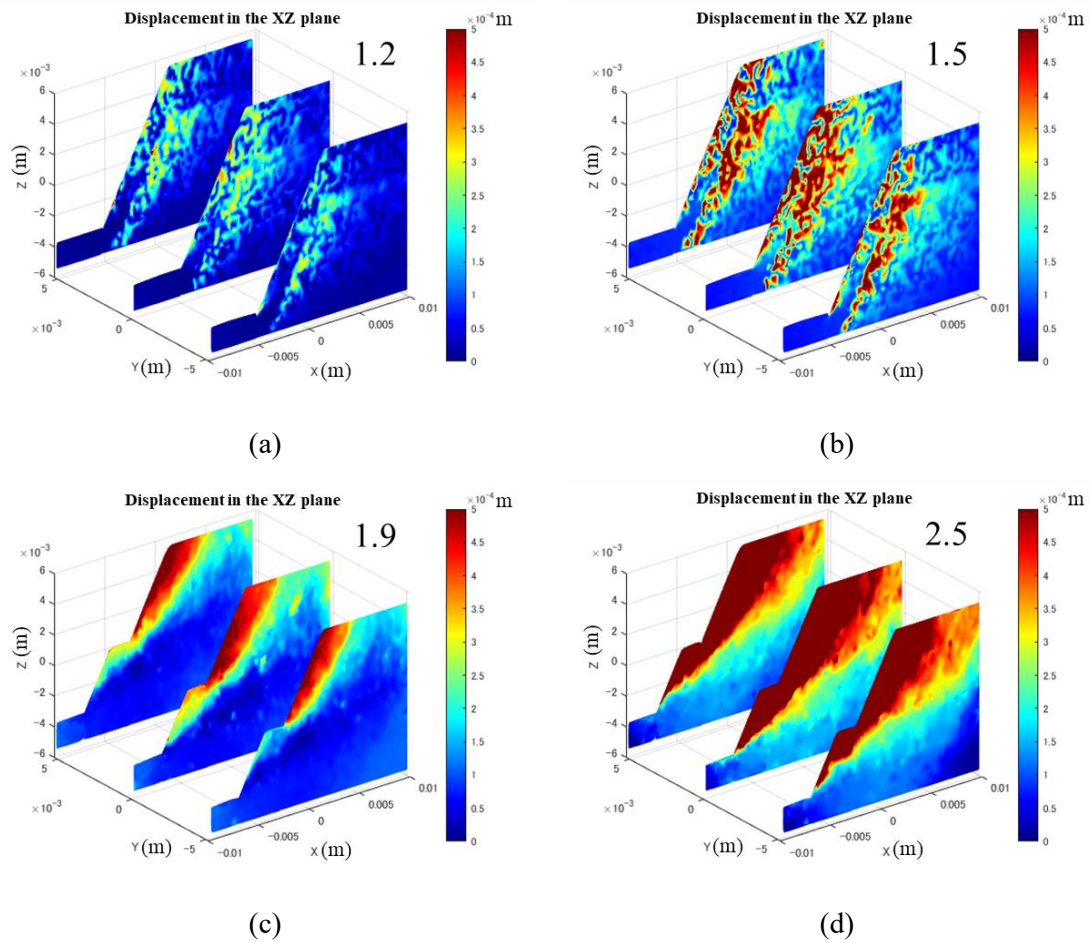


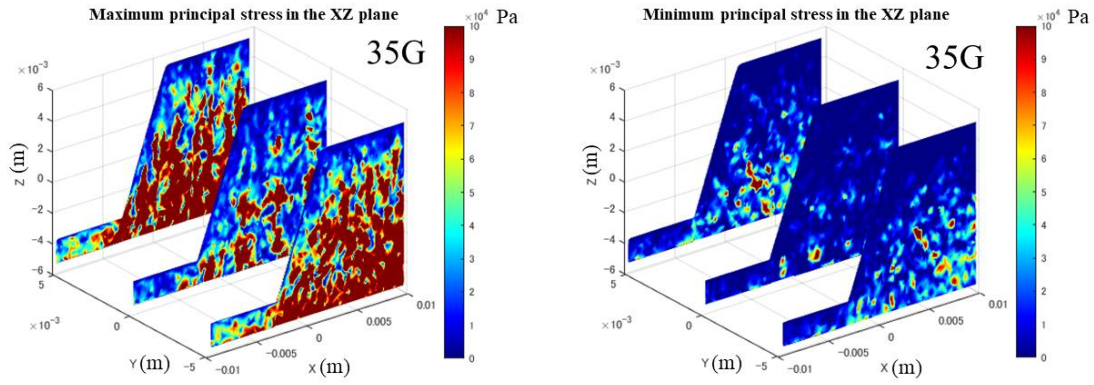
Fig. 8-18 Contour slices for displacement distribution in slopes with varying configurations subjected to ground motions at 16.50 s: (a) uniform slope (55°) with a magnification of 1.2; (b) uniform slope (55°) with a magnification of 1.5; (c) benched slope with a magnification of 1.9; (d) benched slope with a magnification of 2.5

8.4.2 Stress distribution

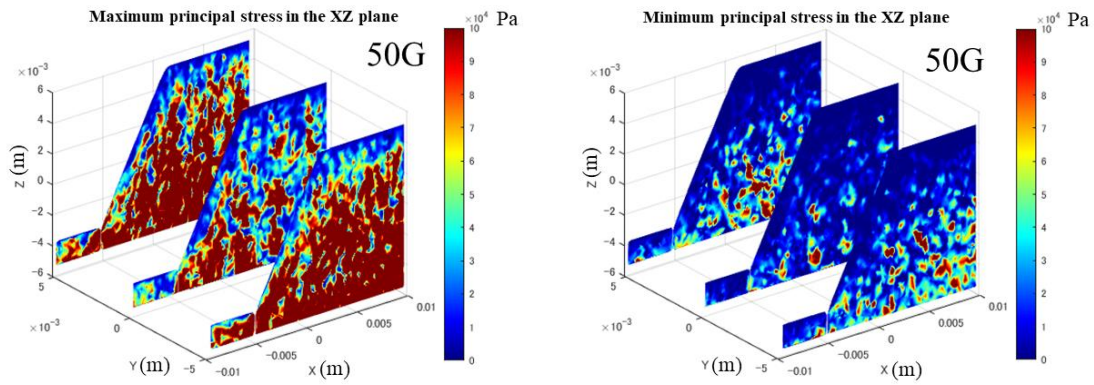
8.4.2.1 Stress distribution affected by the slope configurations under static conditions

The distributions of maximum and minimum stress in the model slopes under static conditions were

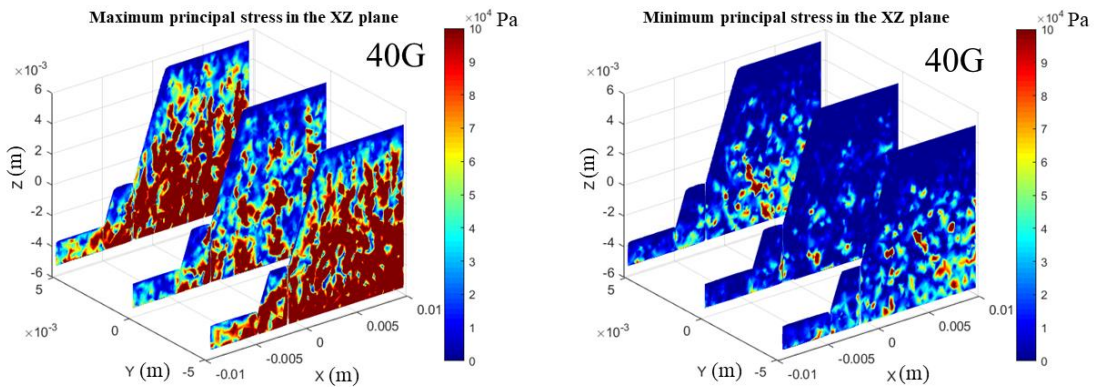
presented in **Fig. 8-19**. Because most volume of the slope was subjected to compression, the lower range of the legend was set to 0. The level of the compression and the volume compressed increased with increasing gravitational force. The compression pressure decreased with height. The section near the top and surface of the slope was either in equilibrium or under tension. Additionally, there are tensile areas scattering in the compression district, implying the transition of force is closely affected by the essence of the particle structure of the model. It will be illustrated by the force chains in **Section 8.4.6**.



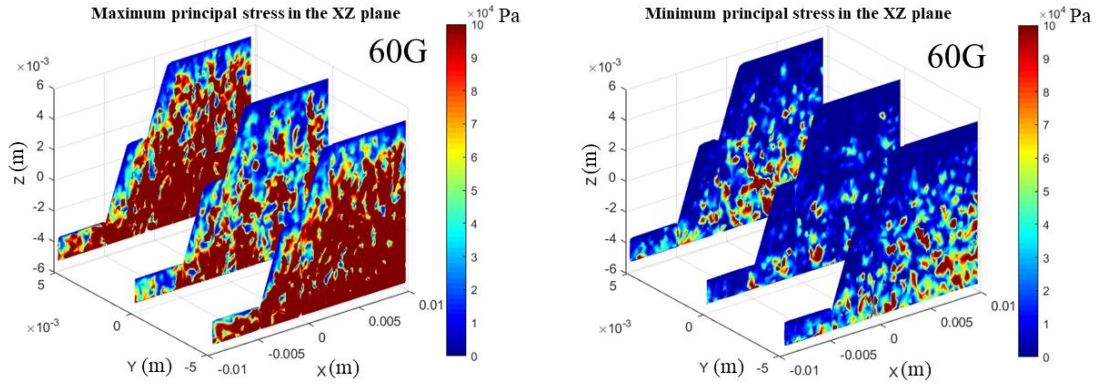
(a)



(b)



(c)



(d)

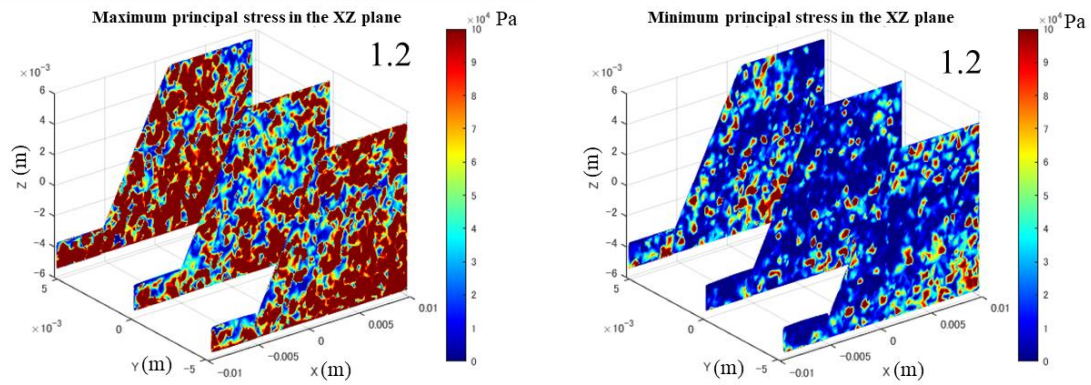
Fig. 8-19 Contour slices for distribution of maximum and minimum principal stress in slopes with varying configurations before sliding: (a) uniform slope (65°) at 35G; (b) uniform slope (55°) at 50G; (c) benched slope (55°) with a low first bench at 40G; (d) benched slope (55°) with a high first bench at 60G

The center slice showed less compression than the two boundary slices, indicating an arching phenomenon in the slope body due to the lateral resistance. It is verified by the previous displacement distribution, which showed a larger magnitude at the center of the model.

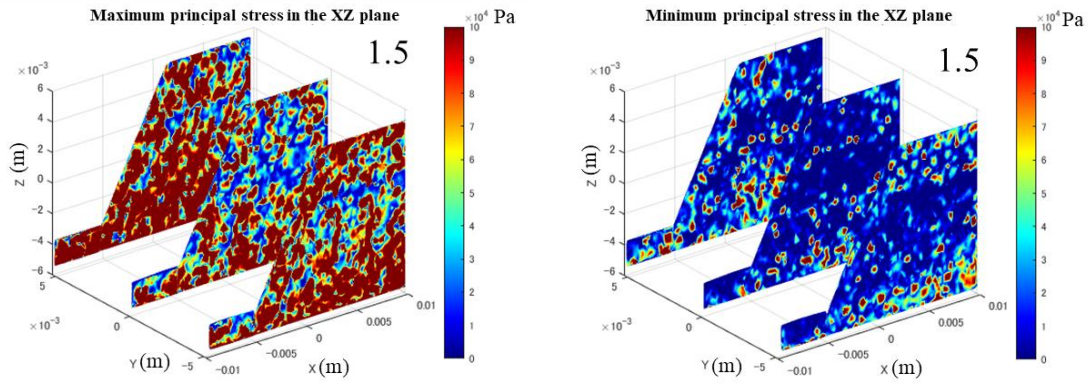
8.4.2.2 Stress distribution affected by the slope configurations subjected to ground motions

In **Fig. 8-20**, the stress distributions in the uniform slope at 16.05 s and benched slope models at 15.59 s subjected to ground motions were presented. The reason was explained in **Section 8.4.1.2**.

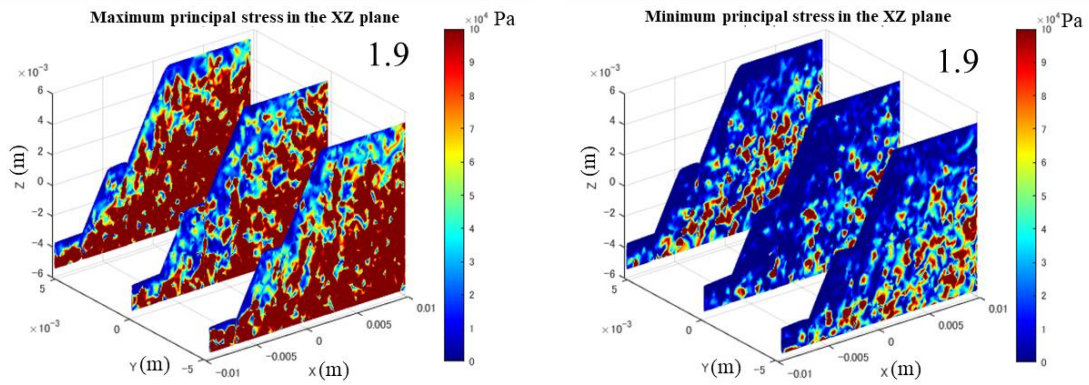
For each type of slope, the compression level and volume were increased with the input wave's amplitude. However, the compression was more pronounced in the uniform slope, even they were subjected to less intensified seismic excitation than the benched slope. This means that the additional first bench shared the external force with other slope part.



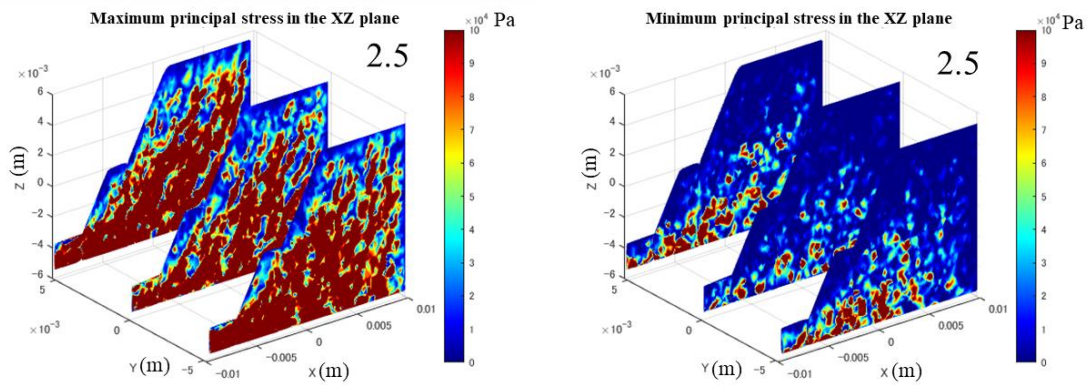
(a)



(b)



(c)



(d)

Fig. 8-20 Contour slices for distribution of maximum and minimum principal stress in slopes with varying configurations subjected to ground motions at 16.50 s: (a) uniform slope (55°) with a magnification of 1.2; (b) uniform slope (55°) with a magnification of 1.5; (c) benched slope with a magnification of 1.9; (d) benched slope with a magnification of 2.5

8.4.3 Comparison of the stress distribution between the simulation and the centrifugal tests under static conditions

Following the configuration of sensors in the centrifugal test, a group of measurement spheres were

placed in the DEM model, measuring stress at corresponding directions under static conditions. Their configurations are shown in **Fig. 8-21**.

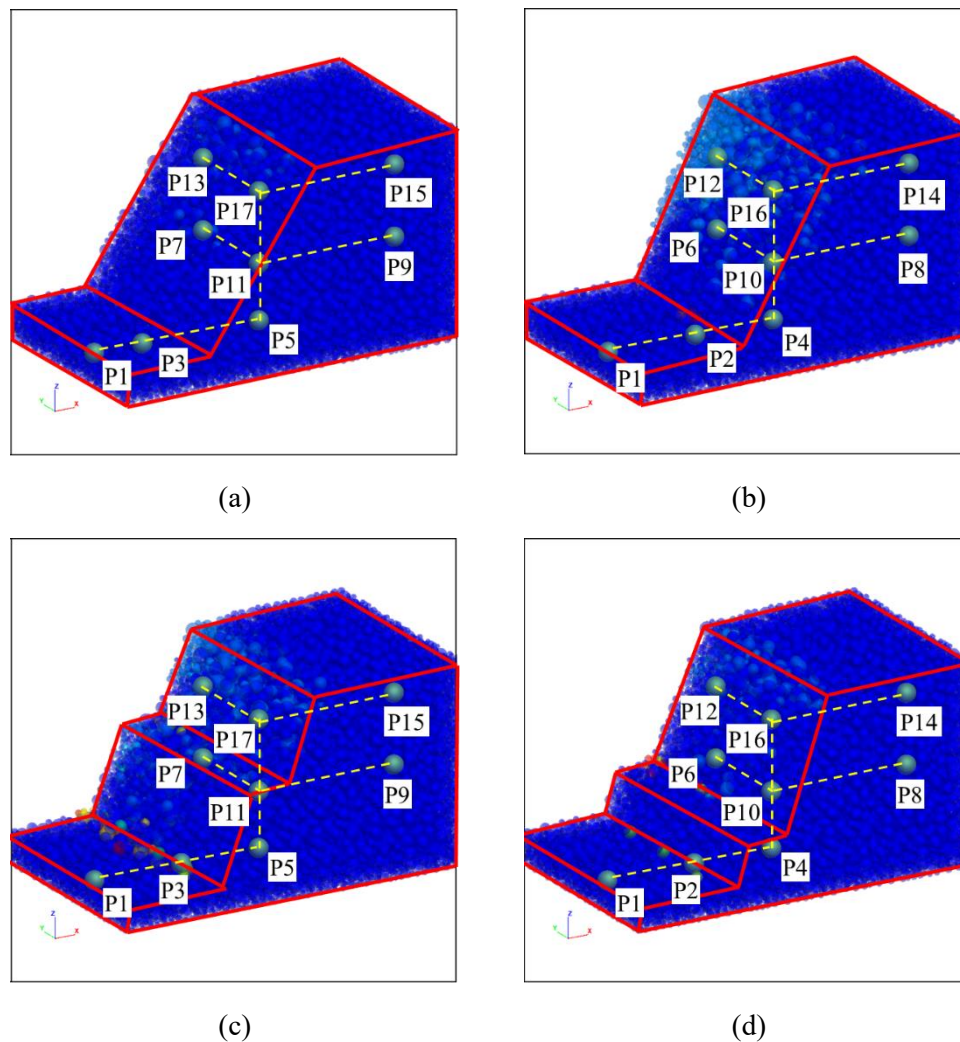


Fig. 8-21 Configurations of the measurement spheres: (a) uniform slope (55°); (b) uniform slope (65°); (c) benched slope (55°) with a high first bench; (d) benched slope (55°) with a low first bench

The measured earth pressure in the simulation was compared with the centrifugal test. Case 2 in **Chapter 4** and Case 4 of the first group in **Chapter 5** were selected for comparison with the uniform and benched slope models, respectively. The model geometries in **Fig. 8-21 (a), (b), (c) and (d)** were the same as the left slope in Case 2, right slope in Case 2, left slope in Case 4 and right slope in Case 4, respectively.

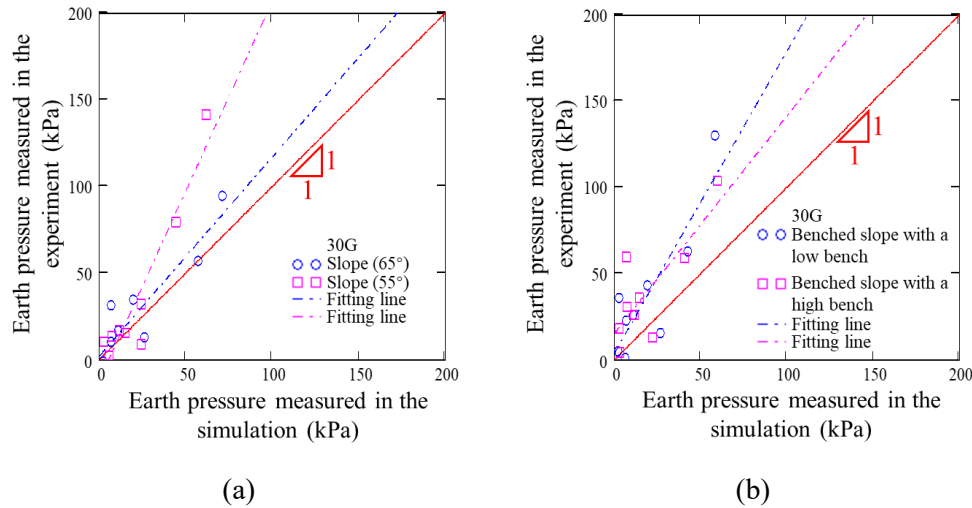


Fig. 8-22 Comparison of earth pressure between simulations and centrifugal tests

Comparison between the simulated and experimental earth pressures under static conditions are shown in **Fig. 8-22**. The comparison was conducted for 30G when all slopes are still stable. The values from the experiments are generally larger than that by the simulation. It is because in the simulation model had decreased dimensions, the diameter of the measurement sphere also reduced; hence, number of particles accommodated in the sphere was not enough, leading to instability of measured values. Another possible reason is the compaction process in the experiments could induce higher earth pressure, while the foundation of the simulation was produced at one time. However, the simulated results and the experimental results were still in the same order of magnitude, indicating the validity of the simulation.

The reduction factors of each slope in the simulation were also calculated as **Table 8-3**. All slope models were around the limit active states where $r_d = -1$, which were similar to the experimental results. Moreover, values for the benched slopes were higher than the uniform slopes when they have the same inclination, implying the benched slopes experienced less deformation under static loadings

Table 8-3 Calculation of reduction factor based on the simulation

	Uniform slope		Benched slope	
r_d (P7, P11)	55°	-0.42	65°	-0.44
r_d (P9, P11)		-1.35		-1.27
r_d (P13, P17)		-0.64		-0.9
r_d (P15, P17)		-1.00		-1.03
r_d (P6, P10)	65°	-0.56	65°	-0.34
r_d (P8, P10)		-1.67		-1.34
r_d (P12, P16)		-1.03		-0.71
r_d (P14, P16)		-1.46		-1.31

8.4.4 Comparison of the deformation between the simulation and the centrifugal tests under static conditions

Particle number 100355 (see **Fig. 8-23**) on the crest of all simulated models was selected to compare with the displacement recorded by the laser sensors in the experiments. From **Fig. 8-24**, the simulation model has a larger displacement than the centrifugal model.

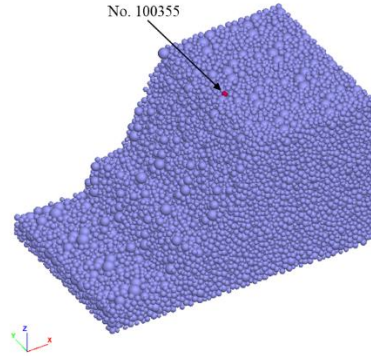


Fig. 8-23 Target particle for the measurement of displacement of the slope crest

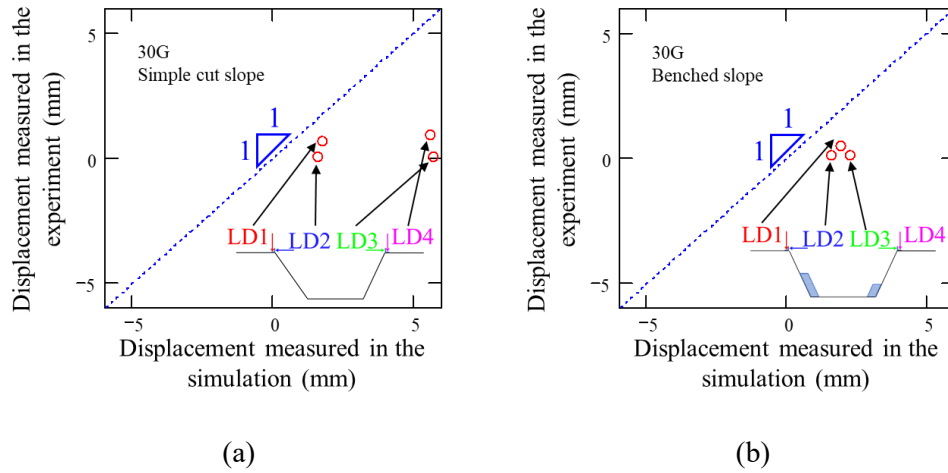


Fig. 8-24 Comparison of displacement between simulations and centrifugal tests: (a) uniform slope; (b) benched slope

8.4.5 Variation of coordination number

The coordination number, which represents the average number of contacts each particle has, is a crucial microstructural index of granular materials and acts a significant role in particle-scale kinematics and microscopic mechanisms. The contact grid structure, represented by the coordination number in a granular system, reflects the packing tightness and structural strength (Xu et al., 2021). It can characterize the packing intensity, a crucial aspect of the material fabric. It is also linked to volumetric change, displaying the relationship between macro-scale deformation and micro-scale structure (Zhang et al., 2019).

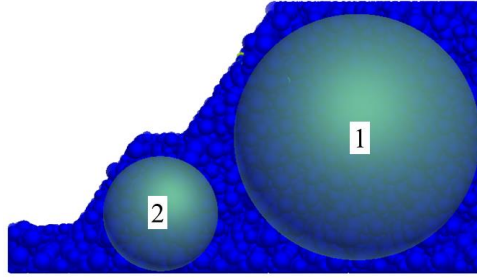


Fig. 8-25 Configuration of measurement spheres for determining the coordination number

For the dynamic case of benched slopes discussed in **Section 8.3.2.2**, the variation in coordination numbers across different parts of the model under varying seismic wave intensities was analyzed. **Fig. 8-25** illustrates the arrangement of two measurement spheres in the slope. Measurement Sphere 1 was positioned in the slope region, while Measurement Sphere 2 was located on the first bench.

The measured results are presented in **Fig. 8-26**, showing that the coordination numbers fluctuated during shaking. Initially, the value in the first bench region was higher than in the slope region. When the shaking intensity was low (magnification of 1.9, see **Fig. 8-26 (a)**) and the model remained stable, the coordination numbers increased over time, indicating densification of the slope with continued shaking. In contrast, under higher shaking intensity (magnification of 2.5, see **Fig. 8-26 (b)**), the coordination numbers decreased after sliding occurred at approximately 12.84 s. At the final stage, the coordination numbers increased again. The decrease suggested that the slope experienced dilation during the sliding process.

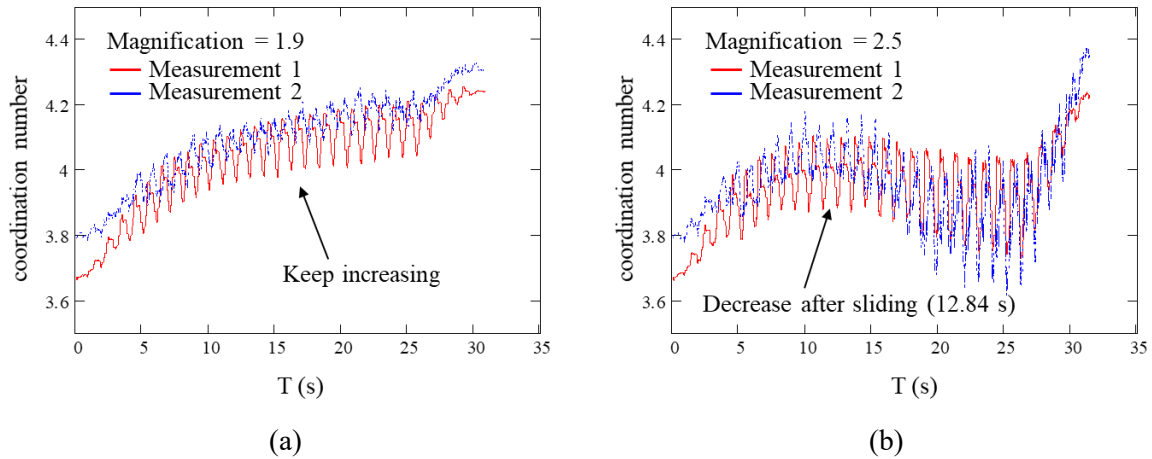


Fig. 8-26 Variation of average coordination number at different regions of the benched slope subjected to the ground motion: (a) magnification of 1.9; (b) magnification of 2.5

8.4.6 Contact force network

8.4.6.1 The force chain arrangement under static conditions

The DEM simulation can reveal particle scale information, one of which is the contact force network (Jin et al., 2024). The force chains are formed by showing all contact forces connecting the centers of

two contacting particles in a particle system. It can be scaled by the force magnitude. From the force chain, we can identify how forces are transmitted through specific particles in a granular system. The force chain reflects the anisotropic characteristics of the system. It can reveal regions with concentrated stresses (force chains) and areas with low stress (void spaces or weak zones). It can help to detect unstable configurations, where force chains are poorly aligned or weakly connected and identify zones at risk of collapse or failure under loading. This information allows engineers to design reinforcement to intercept and distribute forces, preventing localized failure.

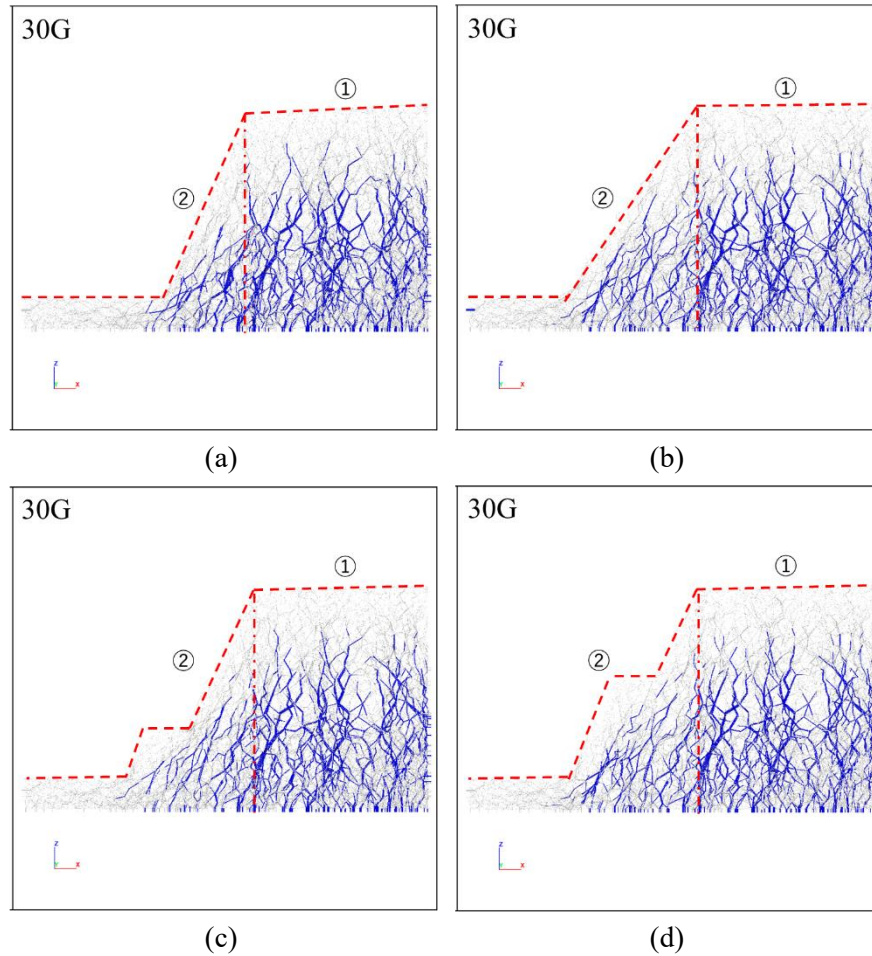


Fig. 8-27 Configuration of force chain networks in slope models with different geometrical configurations under static conditions: (a) uniform slope (65°); (b) benched slope (65°) with a low first bench; (c) uniform slope (55°); (d) benched slope (65°) with a high first bench

In **Fig. 8-27**, the evolution of force chain networks of different slope models was shown with increasing gravitational force. All force chains shown have magnitude larger than 1 N. At 30G, when all models are stable, force chains in area ① are similar for all model configurations, pointing upwards. In area ②, there are more force chains in other configurations than in the slope (65°). All force chains in the first slope are also thicker than that in other slopes, indicating higher stress level is induced in this model. It is because additional force chains in area ② of other models can share the gravitational

force and help to support the slope. In this area, all chains extend from the slope toe to the crest. At the bench part, there are less chains than other places.

8.4.6.2 Force chain's evolution under dynamic conditions

The evolution of the force chains during the shaking are as follows. Only chains with a magnitude higher than 2 N are shown in the figures. For the uniform slope, when the intensity of shaking was low (magnification of 1.2) as shown in **Fig. 8-28**, the main force chains became stronger with shaking time and always pointed form the slope toe to the slope crest. Those weak chains swung with the excitation. At the end of the shaking, more chains accumulated at the slope toe. For a benched slope applied the same shaking wave (see **Fig. 8-29**), at beginning the distribution was similar to the uniform slope, more and more chains were produced at the bench part with the time. At the end, the inclination and the magnitude of chains were smaller than those in the uniform slope.

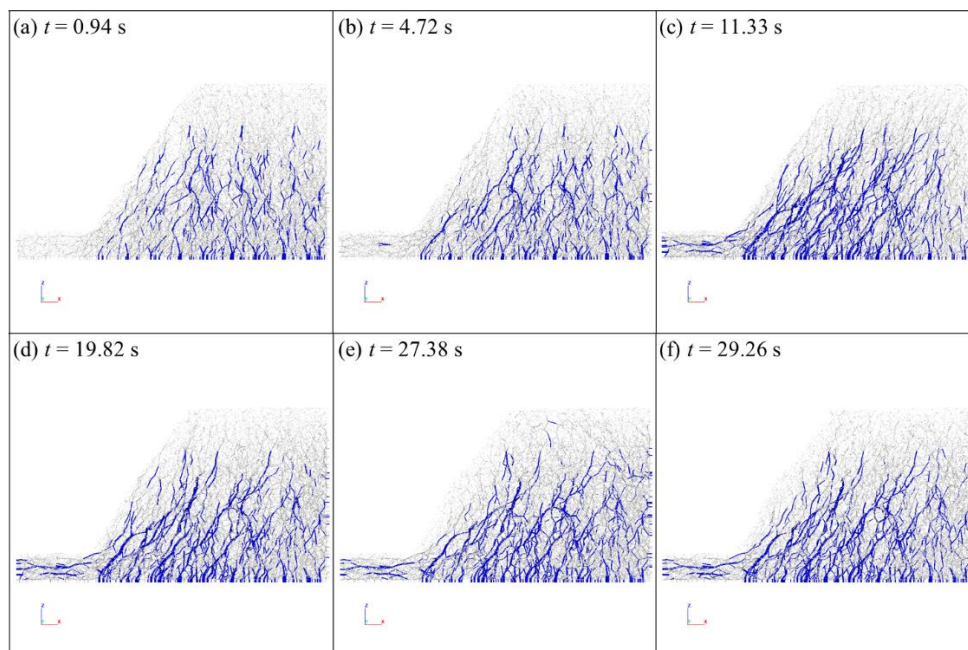


Fig. 8-28 Uniform slope (55°) subjected to shaking with a magnification of 1.2

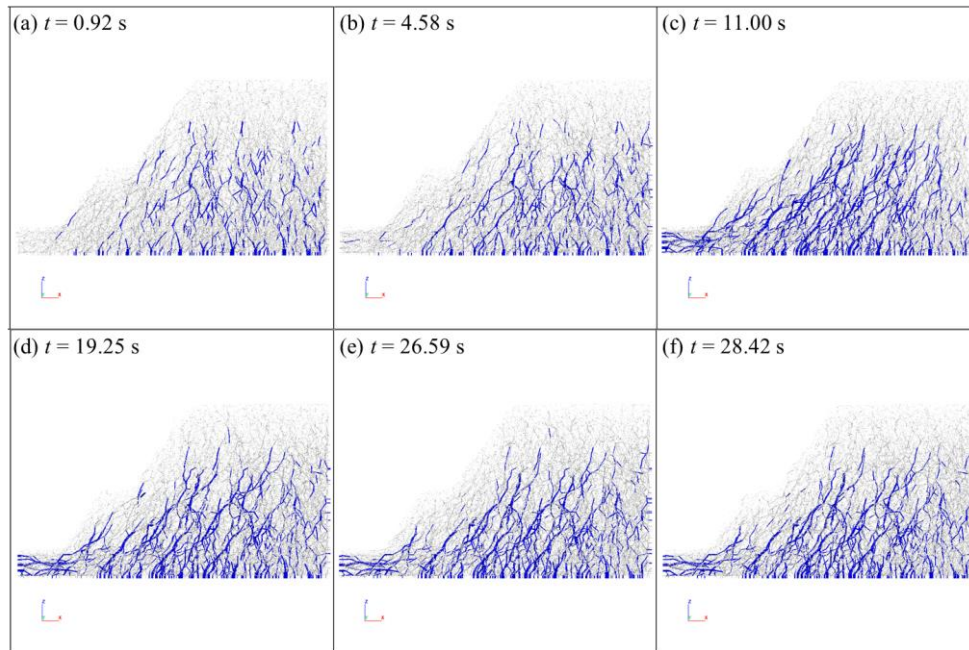


Fig. 8-29 Benched slope (55°) with a high bench subjected to shaking with a magnification of 1.2

When the intensity of the input shaking was increased (magnification of 1.5) to fail the uniform slope model as shown in **Fig. 8-30**, the main force chains were strengthened at the beginning. At 11.33 s (see **Fig. 8-30 (c)**), some chains near the slope surface were bended with the slope deformation, following which, those chains were broken at 19.82 s (see **Fig. 8-30 (d)**) and sliding occurred. However, for the benched slope applied a shaking with a magnification of 2.5, all force chains shook with the wave, and the slope failed when the chains were pointed perpendicular to the slope face (see **Fig. 8-31**).

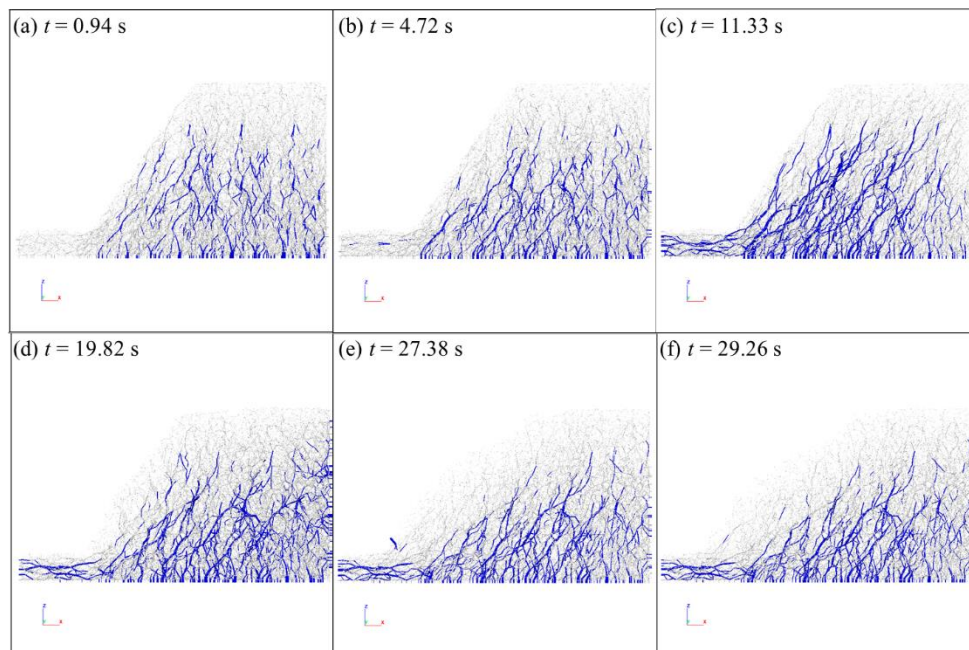


Fig. 8-30 Uniform slope (55°) subjected to shaking with a magnification of 1.5

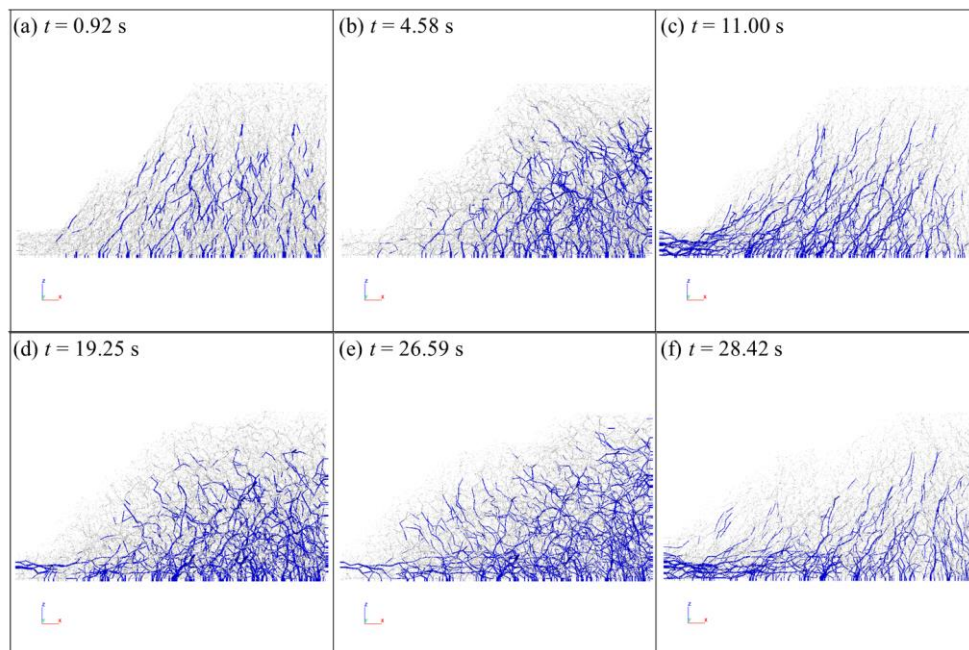


Fig. 8-31 Benched slope (55°) with a high bench subjected to shaking with a magnification of 2.5

8.5 Conclusions

In this chapter, 3D DEM simulations were conducted to study the static and dynamic stability of benched slopes. Due to limitations in centrifugal testing, the sliding surface and full stress and strain distributions cannot be directly obtained. DEM, however, effectively simulates large deformation processes and captures detailed slope sliding behavior. Model parameters were calibrated using direct shear tests to determine contact properties, and simulations of both dry and humid sand agreed well with experimental data. For the benched slope simulations, various configurations were tested and compared to uniform slopes. The sliding process, including the initiation area, sliding surface, and final state, was fully captured. Additionally, particle-level information was provided through force chain networks, offering deeper insights into the instability mechanisms of benched slopes. The main conclusions are listed as follows.

(1) DEM simulations can achieve good consistency with experimental results by applying the principles of centrifugal modeling, reducing the model size, and increasing the gravitational force. This approach maintains a similar stress level in the simulation to that of the real model, while allowing the use of particles with comparable size to the real sand and controlling the number of particles.

(2) Compared to a uniform slope, a low first bench provides additional resisting force at the slope toe, altering the failure mode from toe sliding to sliding at the upper bench. As the first bench's height increases, the failure possibility at both benches becomes more comparable.

(3) For the slope with a high first bench, different failure modes were found in the experiment and the simulation. This discrepancy may be attributed to simulated model's more homogeneous material properties. In centrifugal model slopes, the failure mode was influenced by the water content distribution and the change of centrifugal loading with height.

(4) The failure plane of the laterally confined slope formed a 3D curved surface, with a deeper profile at the center of the slope and shallower areas near the boundaries. Localized failure near the boundary, observed in the experiment, did not occur in the simulation, suggesting that the compaction level near the walls in the experiment was lower than in other areas.

(5) In both static and dynamic cases, deformation initiated at the slope crest. As the deformed area extended to the slope toe or bench toe, sliding was likely to occur. This suggests that the upper slope is the weakest area and may require retrofitting.

(6) The force chain pattern in the uniform slope under static conditions can be categorized into two types. Force chains directly beneath the slope top are oriented upward, while those near the slope surface gradually rotate to align to the slope face. With an additional bench, force chains extend further to reach the bench toe, providing additional resisting force to the slope and distributing the stress induced by external loading.

(7) For the benched slope, at the onset of dynamic excitation, most force chains were concentrated in the slope area. As shaking continued, an increasing number of chains formed within the bench region. Under low-intensity dynamic excitation, force chains could be classified into main and subgrade chains. The main chains retained their orientation throughout the shaking, with their magnitude gradually

increasing, while subgrade chains shifted direction periodically with the shaking. Sliding could occur when the main chains fractured. Under strong shaking, all force chains exhibited oscillations, and failure was more likely when their orientations aligned perpendicularly to the slope surface.

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Chapter 9 Impact of shear pin alignments on seismic retrofitting of benched slopes in centrifugal model tests

Due to the changing topographical conditions resulting from excavation, permanent reinforcements such as anchors and shotcrete are not suitable for engineering sites, such as open-pit mines. Instead, small piles can be used as shear pins to temporarily stabilize the slope, offering the advantages of low cost and ease of installation. These piles can effectively delay the slope failure process, providing additional time for evacuation and thereby reducing potential casualties and economic losses.

In this chapter, the benched slopes' seismic stability affected by shear pins was investigated through centrifugal model tests.

Section 9.1 provides background on previous studies related to shear pins.

Section 9.2. presents various configurations of shear pins installed on the benched slope. The shear pins can be inserted on the slope face or at the crest and can be arranged in rows, columns, or a matrix. The study also examines the effects of pin quantity and spacing.

Section 9.3 describes the recorded horizontal seismic coefficients at the onset of shallow sliding, as well as the observed differences in failure modes among model slopes with various shear pin configurations. Additionally, sensor data is presented to illustrate variations in earth pressure and slope displacement during shaking.

In **Section 9.4**, the experimental results are compared with various assumptions regarding the mechanism by which shear pins enhance slope stability. The state of the slopes, evaluated using the reduction ratio r_d as shaking magnitude increases, is also discussed.

Lastly, the findings of this part are listed in **Section 9.5**.

9.1 Background

Many slope stabilizing approaches have been proposed so far, involving many kinds of instruments and structures. Soil nails are one of the mature methods to reinforce cohesive slopes. The inclination and length of the nails can affect the effectiveness, which were verified by the Finite difference method (FDM) (Singh et al., 2021). The existence of an optimal incline angle of the soil nail was also proved by parametric analysis of the derived results based on a 3D discrete failure mechanism (Chen et al., 2022). Piles are another useful way by which the driving force of the shallow layer can be transferred to the deep layer. Explicit-finite-difference code (FLAC 3D) with an internal routine (FISH) was used to conduct a coupled analysis for a slope-pile system, revealing the change of the critical slip surface caused by piles (Won et al., 2005). Sleeved piles can reduce the normal and shear stress in front of piles, thus increase the FS for the slope compared to piles without sleeves (Oliaei & Tohidifar, 2018). Excavation at the slope toe can lead to stress redistribution and undermine the slope stability, which can be remedied by a pile-anchor structure (Xu et al., 2021), shaking table tests were also conducted to observe the function of piles in this situation, good seismic performance of a slope reinforced by anchored piles was demonstrated (Hu et al., 2021). Besides, many researches have paid attention to using pre-stressed cables to help piles resist large bending moment (Chen et al., 2016) or inserting the anchor cables directly into the bedrock for slope reinforcement. Compared to soil nails, the anchor cable can exert the strength of soil better, but it is prone to fracture. These methods are often combined with the retaining wall. With the increase of seismic load, the proportion of total sliding force borne by each part of a pile- cantilever retaining wall (CRW) combined structure changed, reflecting the deformation trend of the slope under seismic excitation (Sun et al., 2022). An innovative retaining wall founded on barrettes stabilized by prestressing cables was constructed for a deep cut in a slope (Rabaiotti & Malecki, 2018).

Retrofitting benched slopes is a common challenge in open-pit mining. The previously mentioned methods are often not cost-effective and may not adapt well to continuously changing site conditions. Therefore, small stabilizing piles (shear pins) offer a practical solution, as they can effectively delay the slope failure process and are easy to install. To assess the impact of small piles, soil blocks resting on a low-friction interface plane stabilized by brittle shear pins were investigated through 1G testing (Ukritchon et al., 2017) and numerical simulation (Ukritchon et al., 2019). The soil blocks' static stability was affected by the numbers and placement of shear pins. However, optimizing these two factors for the benched slope with complex geometry under dynamic conditions is more difficult. Hence, seismic centrifuge tests have been carried out to observe the benched slope's seismic resistance with shear pins.

9.2 Experimental scheme

9.2.1 Test program

Centrifugal model tests were carried out to study the dynamic stability of benched slopes retrofitted

with shear pins subjected to ground motions. The centrifuge facility utilized for these tests is located at the Geotechnical Centrifuge Center, Disaster Prevention Research Institute (DPRI), Kyoto University, featuring an effective radius of 2.5 m, maximum payload of 24 G×ton and a maximum centrifugal acceleration of 200G (Tao & Pipatpongsa, 2022). The attached shaking table to the centrifuge has a maximum capacity of 142.5 kg at 50G for dynamic excitation.

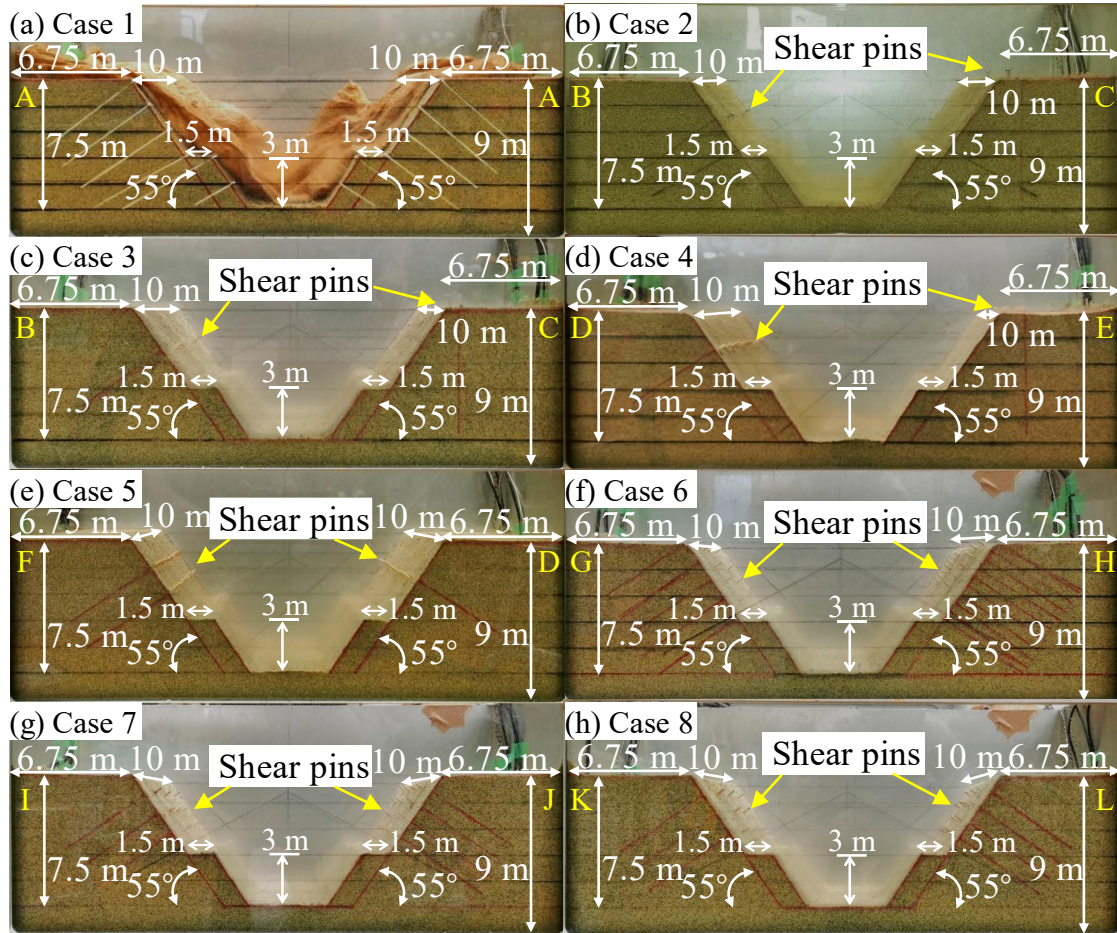


Fig. 9-1 Established model slopes before loading with different alignments of shear pins (prototype scale at 50G)

Eight model slopes with twelve arrangements of shear pins were prepared in this study. For each case, two model slopes were tested simultaneously with symmetric seismic excitation to increase efficiency and to facilitate the observation of the effects of different retrofitting methods. **Fig. 9-1** illustrate built model slopes before loading, with all model slopes 9 m in a total height and 7.5 m in a slope height on the prototype scale at 50G. The alignment pattern of shear pins on each model slope is labelled by capital letters. According to the previous study, benched slopes are prone to failure primarily on the upper bench due to seismic amplification. Thus, only the upper part of the benched slope was installed with shear pins. **Fig. 9-2** shows the diagrams of all corresponding patterns. Black points represent the location of shear pins. In models A to F, shear pins are set up in rows, either perpendicular to the slope

surface or to the crest. In models G and F, they are arranged in columns on the slope surface. In model I to L, they are aligned in a matrix on the slope surface.

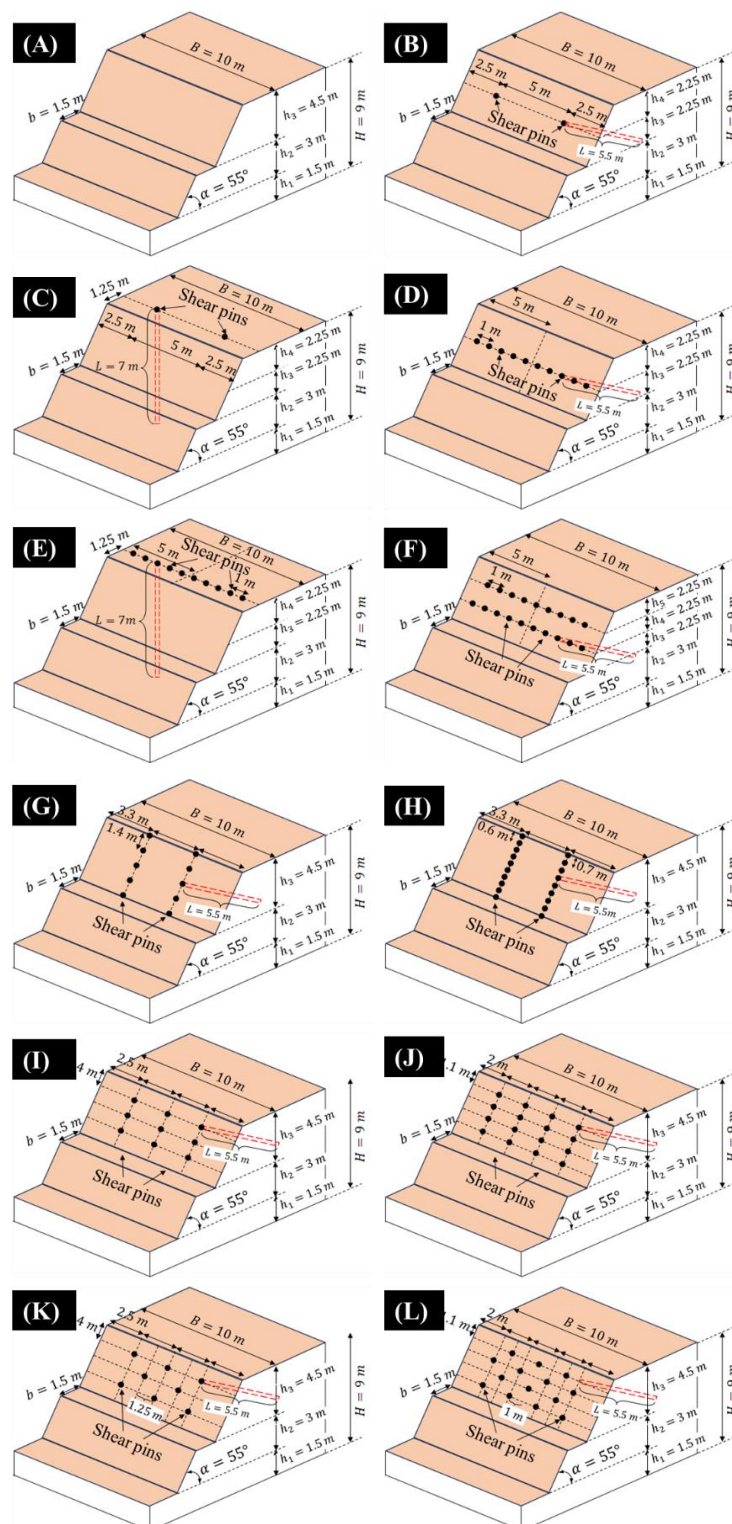


Fig. 9-2 Alignment patterns of shear pins for centrifuge tests

Metal and wooden shear pins with diameters of 0.175 m and 0.138 m, respectively, in prototype were

utilized as depicted in **Chapter 3**. Both materials had a longer type and a shorter type with lengths of 7.5 m and 6 m, respectively. These pins were inserted into the slope crest with a depth of 7 m and into the slope surface with a depth of 5.5 m. All cases were built with damp Hiroshima sand under $D_c=90\%$ and $w=8\%$. Dry udon noodles were inserted into the model in Case 1 to visualize the sliding surface. Only the narrow soil chamber is used for all cases in this chapter (see **Chapter 4**).

Table 9-1 and **Table 9-2** show the detailed model configurations for eight cases, including the case number, experimental date, type of soil chamber used, degree of compaction (D_c), slope inclination (β), the first bench's height (B_H) and width (B_W), initial water content (w), the centrifugal loading, number of shear pins installed and the sequence of amplitude magnifications (which were explained in **Chapter 6**) in each test. A centrifugal loading of 50G was employed in the tests. A diagram illustrating model slopes under centrifugal loading for two groups is shown in **Fig. 9-3**.

Table 9-1 Model configurations for benched slopes subject to ground motions

No.	Date	Soil chamber	Degree of compaction, D_c (%)	Slope angle, β (°)		1 st bench height, B_H (m)		1 st bench width, B_W (m)		Initial water content, w (%)	Loading (G)
				Left	Right	Left	Right	Left	Right		
1	221019	Narrow	90	55		3		1.5		8	50
2	230524										
3	230526										
4	230718										
5	230720										
6	230821										
7	230823										
8	230825										

Table 9-2 Number of shear pins and amplitude magnification in the tests

No.	Date	Number of shear pins		Shaking magnification	Remark
		Left	Right		
1	221019	0	0	1.9→2.5→2.7	original slope
2	230524	2	2	2.2→2.5→2.7→3.0	steel pins, aligned in rows
3	230526	2	2	2.2→2.5→2.7	wooden pins, aligned in rows
4	230718	10	10	2.2→2.5→2.7→3.0→3.3	wooden pins, aligned in rows
5	230720	19	10	2.2→2.5→2.7→3.0	wooden pins, aligned in rows
6	230821	10	19	2.2→2.5→2.7	wooden pins, aligned in columns
7	230823	9	16	2.2→2.5	wooden pins, aligned in matrix
8	230825	8	14	2.2→2.5	wooden pins, aligned in matrix

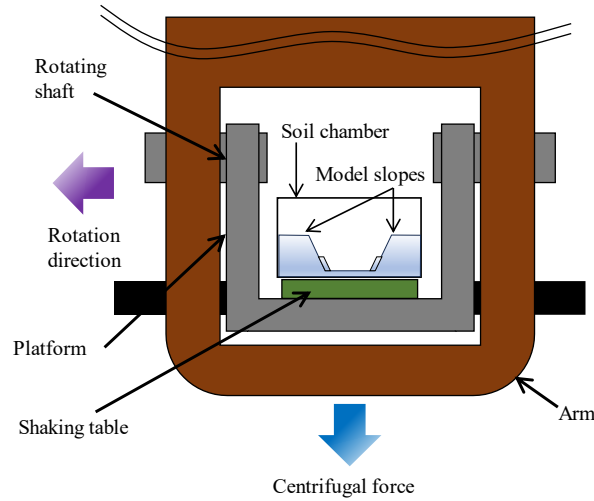


Fig. 9-3 Diagram of model slopes during centrifugal loading

9.2.2 Instrumentation

The specifications of earth pressure transducers and laser sensors adopted in this chapter are the same as those in the **Section 4.2.2**. A maximum of four accelerometers, type A6H-50G, were placed to measure the reaction of the slope models during each seismic excitation.

A maximum of 13 earth pressure transducers were installed inside the model to record the variation of internal earth pressure during each shaking. Additionally, up to six laser sensors were installed to detect the slope's displacements at specific locations.

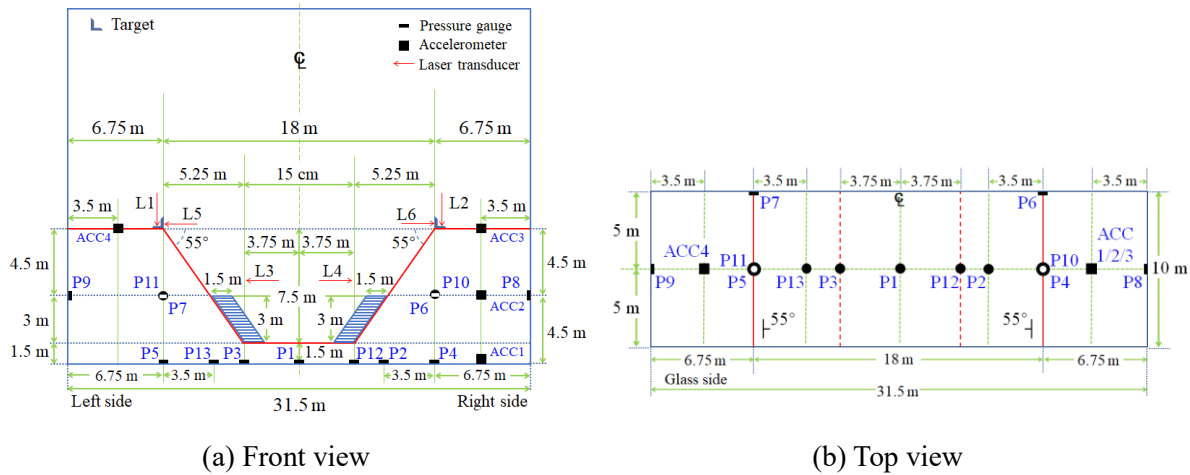


Fig. 9-4 Configuration of sensors in the tests

The arrangement of sensors varied slightly inside each group. Most of the sensors were positioned at the bottom and middle height (see **Fig. 9-4**). Four earth pressure sensors (P6, P7, P8, P9) were placed on the slope boundary to monitor the horizontal earth pressures and other nine earth pressure sensors (P1, P2, P3, P4, P5, P10, P11, P12 and P13) were employed to monitor the vertical earth pressures.

The model slopes' displacements were recorded by laser sensors. Six laser sensors were installed to

record slope crests' vertical and horizontal displacement and the slope surfaces' horizontal displacement at the middle height on both sides. Targets introduced in **Section 4.2.2** were also placed.

Moreover, high-speed cameras were also employed in every case to monitor slope deformation, providing visual evidence to complement the sensor measurements.

The configuration of sensors for all cases is listed in **Table 9-3** and **Table 9-4**.

Table 9-3 List of earth pressure sensors and accelerometers for all cases

No.	Date	Earth pressure sensor													Accelerometer				
		P	P	P	P	P	P	P	P	P	P	P	P	P	ACC	ACC	ACC	ACC	ACC
		1	2	3	4	5	6	7	8	9	10	11	12	13	1	2	3	4	5
1	221019	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	Δ	○
2	230524	○	○	○	○	○	○	○	○	○	○	○	○	○	○	Δ	○	○	—
3	230526	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—
4	230718	○	○	○	○	○	○	○	○	○	○	○	○	○	○	Δ	○	Δ	—
5	230720	○	○	○	○	○	○	○	○	○	○	○	○	○	○	Δ	○	○	—
6	230821	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—
7	230823	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—
8	230825	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	—

Note: ○ The sensor was placed and functioned Δ The sensor was placed but malfunctioned
 - The sensor was not placed

Table 9-4 List of other sensors for all tests

No.	Date	Laser sensor						Moisture sensor					
		LD	LD	LD	LD	LD	LD	W	W	W	W	W	W
		1	2	3	4	5	6	1	2	3	4	5	6
1	221019	○	○	○	○	○	○	—	—	—	—	—	—
2	230524	○	○	○	○	○	○	○	○	○	○	○	○
3	230526	○	○	○	○	○	○	○	○	○	○	○	○
4	230718	○	○	○	○	○	○	○	○	○	○	○	○
5	230720	○	○	○	○	○	○	○	○	○	○	○	○
6	230821	○	○	○	○	○	○	○	○	○	○	○	○
7	230823	○	○	○	○	○	○	○	○	○	○	○	○
8	230825	○	○	○	○	○	○	○	○	○	○	○	○

Note: ○ The sensor was placed and functioned Δ The sensor was placed but malfunctioned
 - The sensor was not placed

9.2.3 Test procedure

The experimental process of each case was similar. Initially, dry Hiroshima sand was blended with water to achieve a desired water content of 8%. Then, prepared sand was divided equally into six portions and compacted into the soil chamber in six layers with a wooden hammer. Each layer was 3 cm in thickness. The weight required for each layer was obtained from the compaction degree ($D_c=90\%$) and the volume of each layer. The sensors were placed when the soil was compacted to their heights. After forming a horizontal foundation with a height of 18 cm, unnecessary soil was excavated from the center, and two slope surfaces were cut out using a spatula. Consequently, the shear pins were inserted into the model slopes by using a rubber hammer. The preparation process is shown in **Fig. 9-5**.

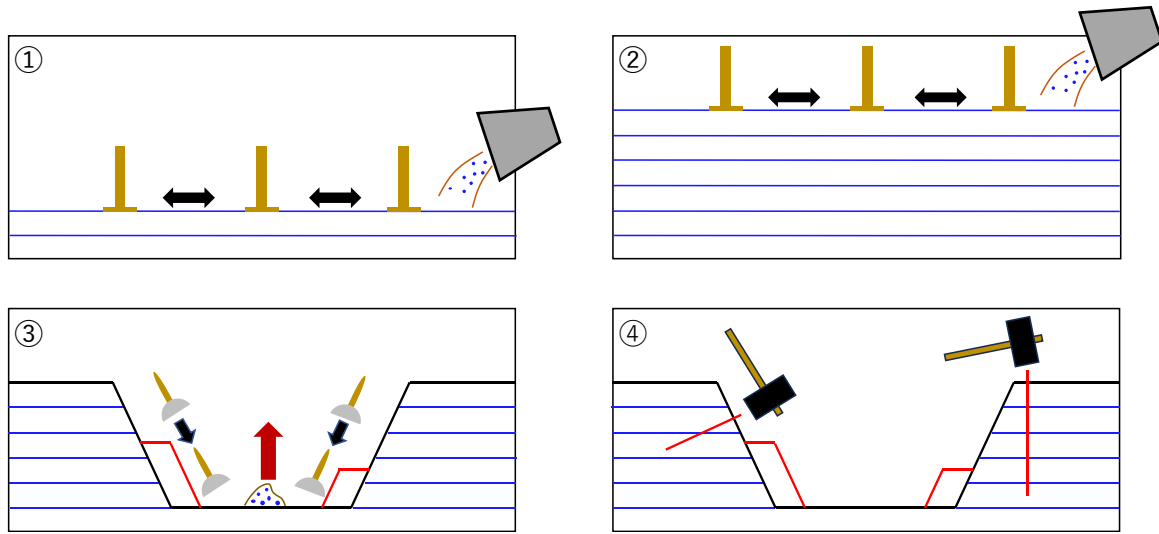


Fig. 9-5 Flow chart of the preparation process

The model was then installed onto the centrifuge platform for testing. The centrifugal loading was initially increased steadily from 1G to 50G first. Then, the centrifugal acceleration remained for a while until values of sensors reached stable. Next, seismic excitations controlled by the displacement were applied to the model in sequence using an inflight shaking table. A datum ramped sine wave as shown in **Chapter 6** was used, with the amplitude being gradually magnified for each input wave. The specific magnification for every case are shown in **Table 9-2**.

9.3 Experimental results

Details of the slope sliding process were captured by the high-speed camera. All sliding events occurred within one second, which will be shown in the time order for each case. The maximum absolute value recorded by the accelerometer placed at the model bottom (ACC1) is selected to indicate each input shaking's intensity. Additionally, the displacement and earth pressure measured by installed sensors are presented as follows.

9.3.1 Case 1 (Number of shear pins: Left, 0, Right, 0; original slope)

Sliding process

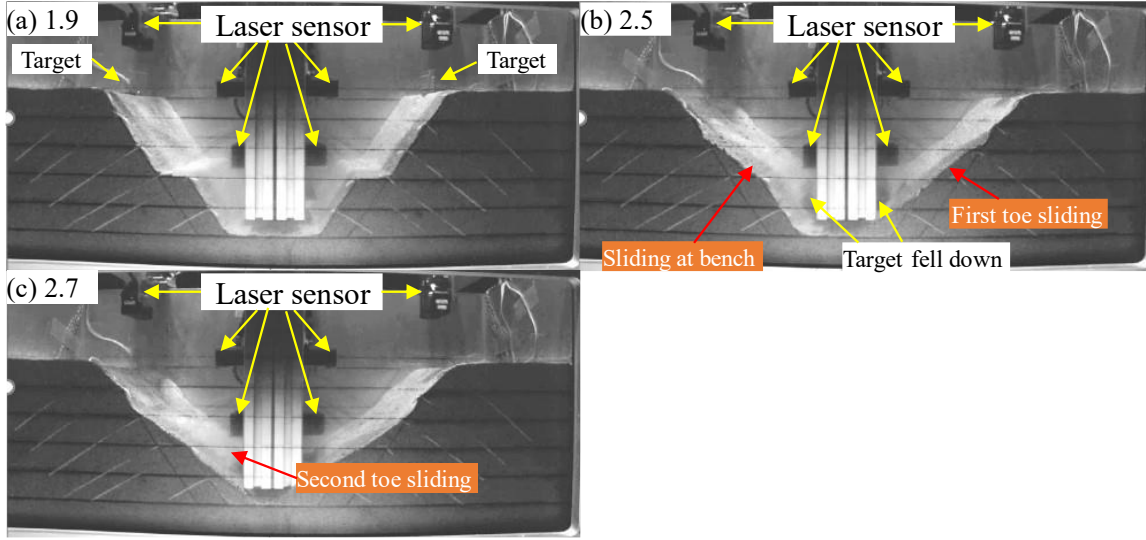


Fig. 9-6 Reaction of the model slopes to each shaking in Case 1: (a) stable state after the first shaking; (b) sliding at left bench and toe sliding of the right slope; (c) toe sliding on the left slope

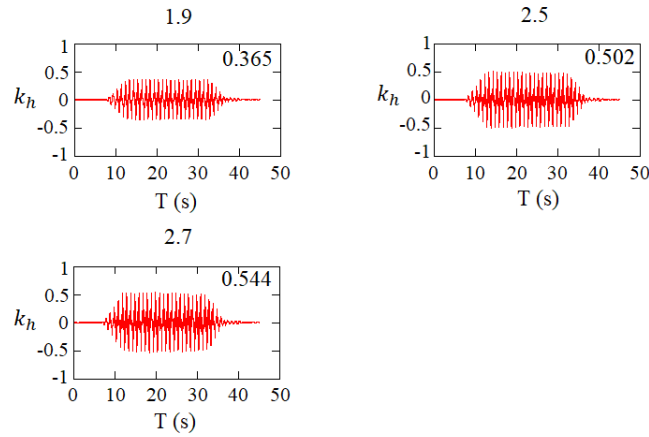


Fig. 9-7 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 1

The definitions of different types of sliding are clarified as follows:

Toe Sliding: The sliding surface connects the slope crest and the toe and across the slope breath.

Local Toe Sliding: The sliding surface connects the slope crest and the toe but only across partial breath.

Bench sliding: The sliding area is confined at bench level.

In Case 1, both model slopes, without the shear pins, remained stable after the first shaking with a magnification of 1.9 ($k_h=0.365$) (see **Fig. 9-6 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.502$), the left second bench failed, and the right slope had a toe sliding (see **Fig. 9-6 (b)**).

During the third sliding with a magnification of 2.7 ($k_h=0.544$) (see **Fig. 9-6 (c)**), toe sliding also occurred on the left slope.

Displacement

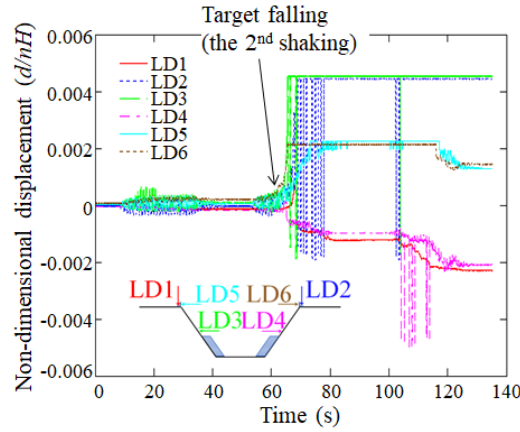


Fig. 9-8 Slope crests' vertical and horizontal displacement of and slope surfaces' horizontal displacement with each shaking in Case 1

The correlation between non-dimensional displacement, d/nH , and each shaking is shown in **Fig. 9-8**. d is the real displacement measured by the laser sensor, $n=50$ is the scaling factor in centrifuge modeling to account for gravity, and H represents the total height of the model, which is 18 cm. Positive values indicate that the target, representing the displacement of the slope crest, is approaching the laser sensor, while negative values indicate that the target is moving away from it. During the second shaking, targets on both crests fell down, all laser sensors either had abrupt changes or went out of range.

Earth pressure

The variation of non-dimensional earth pressure, $P/npgH$, is plotted with respect to each shaking in **Fig. 9-9** for Case 1. P is the earth pressure, and g is the gravitation acceleration, which is typically 9.81 m/s^2 . The earth pressure fluctuates in each shaking and its value can change before and after shaking. The value after each shaking of P1~P5, placed on the model bottom, rise gradually, while other pressures show less variation. P6~P9 barely change between pre-shaking and post-shaking values during the whole process. P10 and P11 have a similar characteristic.

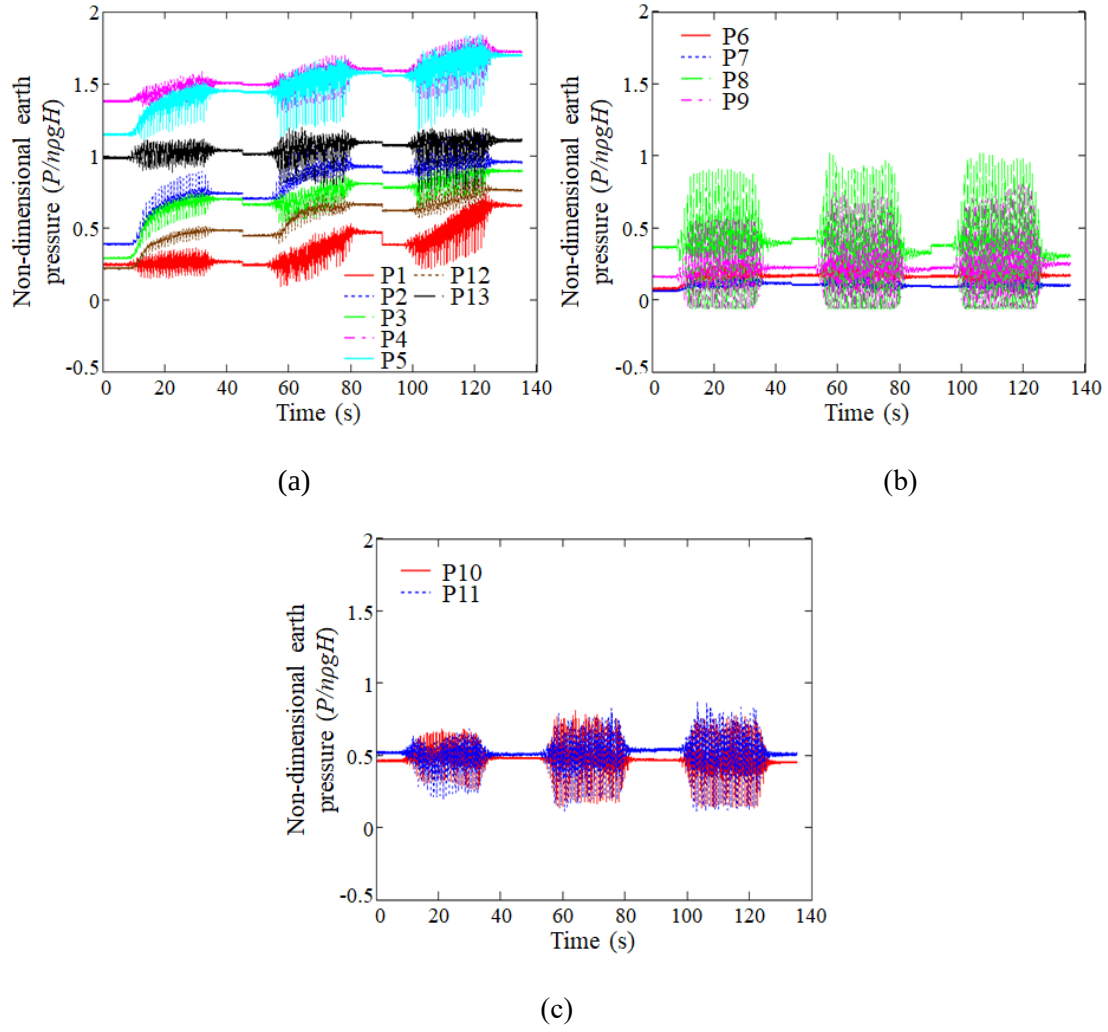


Fig. 9-9 Measured earth pressure with each shaking in Case 1: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

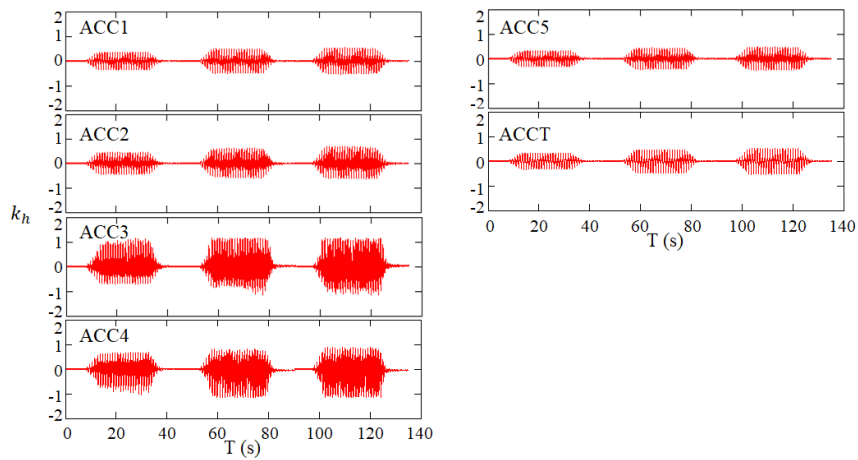


Fig. 9-10 Measured acceleration with each shaking in Case 1

Fig. 9-10 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

9.3.2 Case 2 (Number of shear pins: Left, 2, Right, 2; steel pins, aligned in rows)

Sliding process

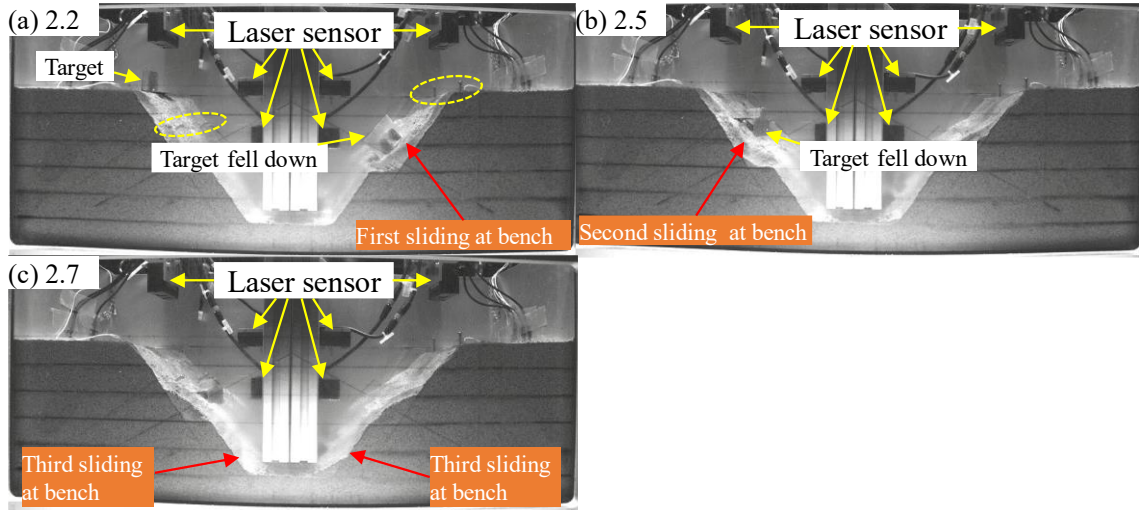


Fig. 9-11 Reaction of the model slopes to each shaking in Case 2: (a) sliding at bench on the right slope; (b) sliding at bench on the left slope; (c) sliding at bench on both slopes

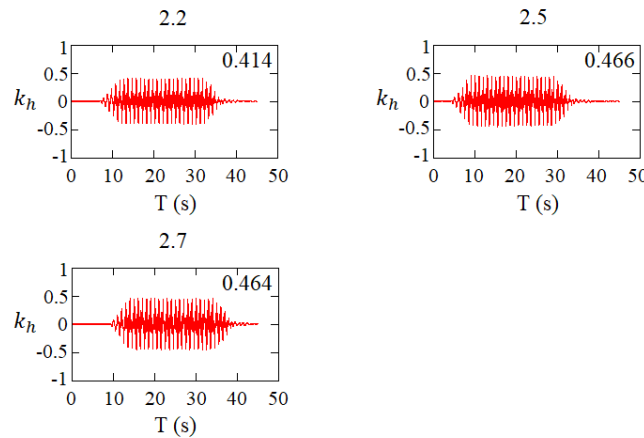


Fig. 9-12 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 2

In Case 2, shear pins were installed on both slopes, with two on the left slope surface and another two perpendicular to the right slope crest, all aligned in a row, as circled in **Fig. 9-11 (a)**. After the first shaking with a magnification of 2.2 ($k_h=0.414$), the second bench of the right slope failed, while the left

slope remained stable (see **Fig. 9-11 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.466$), a sliding happened on the left second bench (see **Fig. 9-11 (b)**). The third shaking with a magnification of 2.7 ($k_h=0.464$), sliding occurred on the first bench of both slopes (see **Fig. 9-11 (c)**).

Displacement

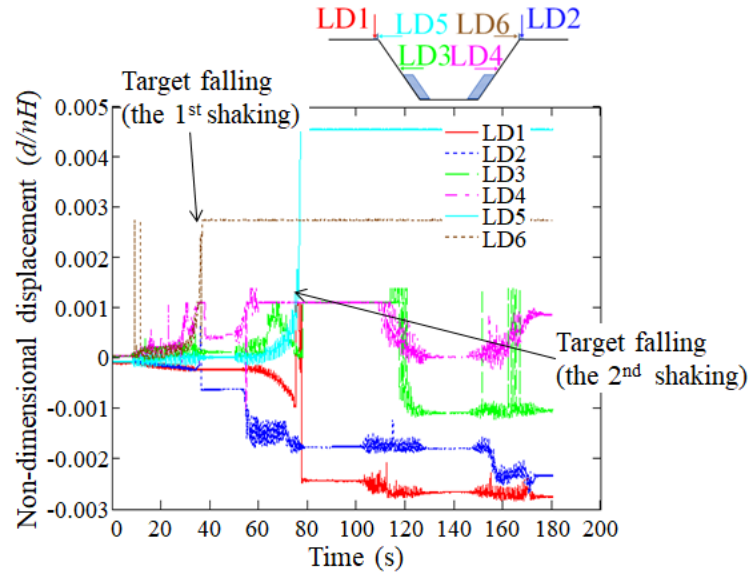


Fig. 9-13 Slope crests' vertical and horizontal displacement and slope surfaces' horizontal displacement with each shaking in Case 2

The symbols in **Fig. 9-13** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 2. The right target fell down in the bench sliding during the first shaking, causing the LD6 to go out of range, LD4 to increase and LD2 to decrease. The second shaking led the left target to fell down, thus, LD5 went out of range and LD1 decreased. Both LD3 and LD4 increased because of the sliding soil.

Earth pressure

Fig. 9-14 presents change of non-dimensional earth pressure during each shaking in Case 2, where the symbols can refer to those in Fig. 9-9.

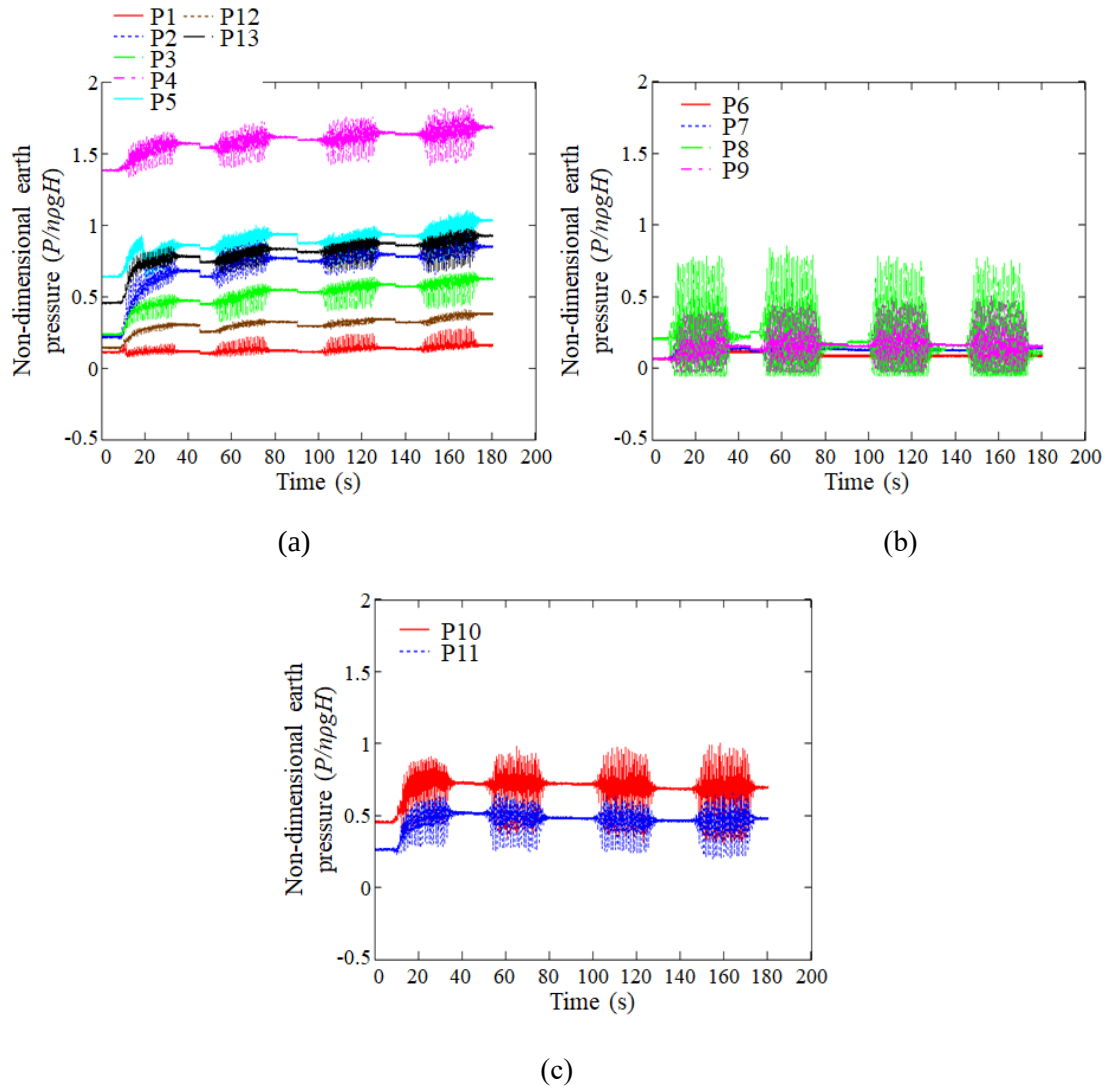


Fig. 9-14 Measured earth pressure with each shaking in Case 2: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 9-15 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

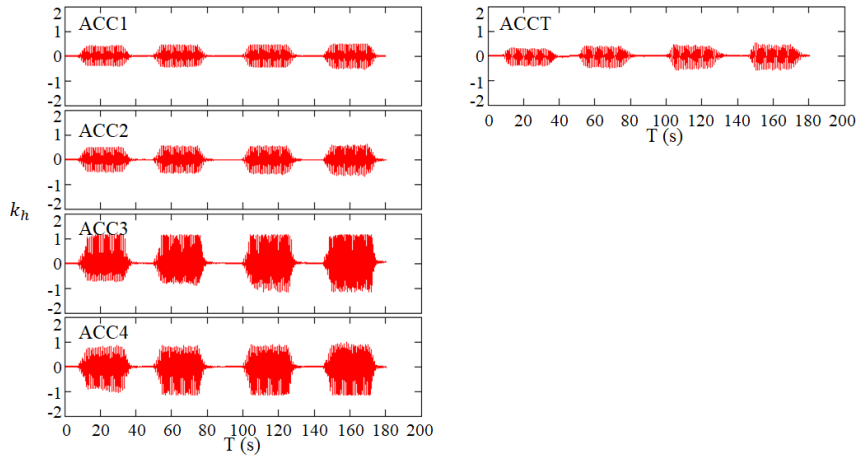


Fig. 9-15 Measured acceleration with each shaking in Case 2

9.3.3 Case 3 (Number of shear pins: Left, 2, Right, 2; wooden pins, aligned in rows)

Sliding process

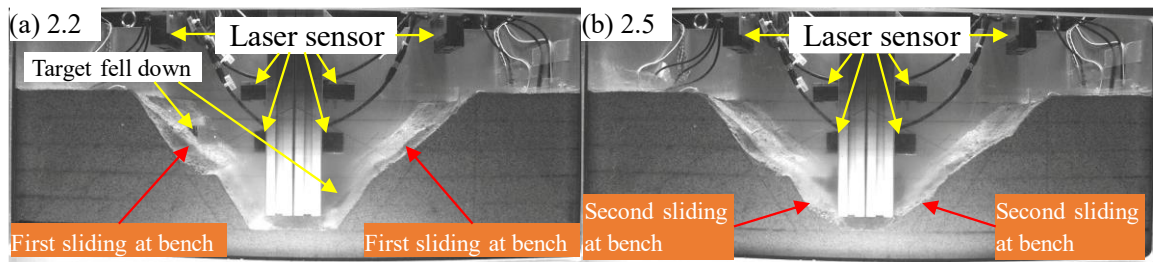


Fig. 9-16 Reaction of the model slopes to each shaking in Case 3: (a) first sliding at bench on both slopes; (b) second sliding at bench on both slopes

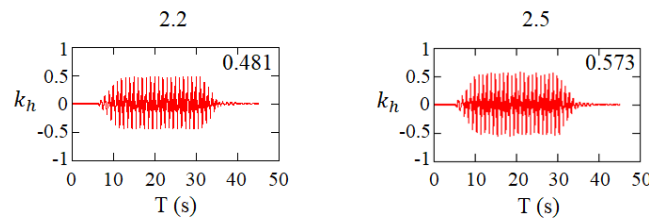


Fig. 9-17 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 3

In Case 3, shear pins were installed at the same locations as in Case 2, but the material was changed to wooden sticks. The second bench of both slopes failed at the first shaking with a magnification of 2.2 ($k_h=0.481$) (see **Fig. 9-16 (a)**). Subsequently, the first bench of both slopes failed at the second shaking with a magnification of 2.5 ($k_h=0.573$) (see **Fig. 9-16 (b)**). As both slopes already failed, the last shaking was not presented here.

Displacement

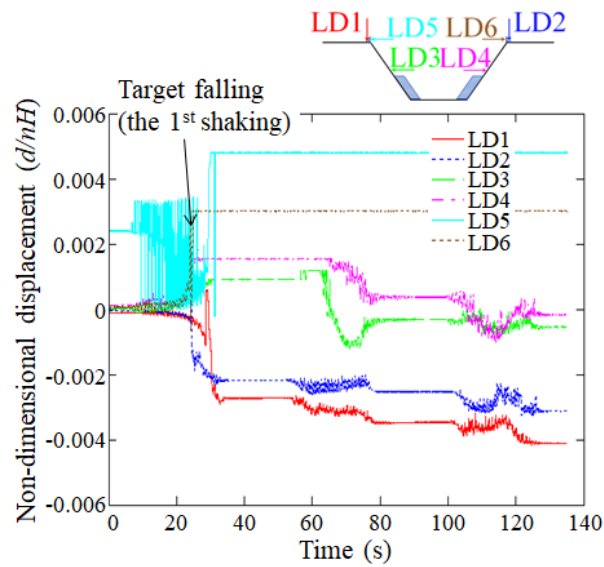
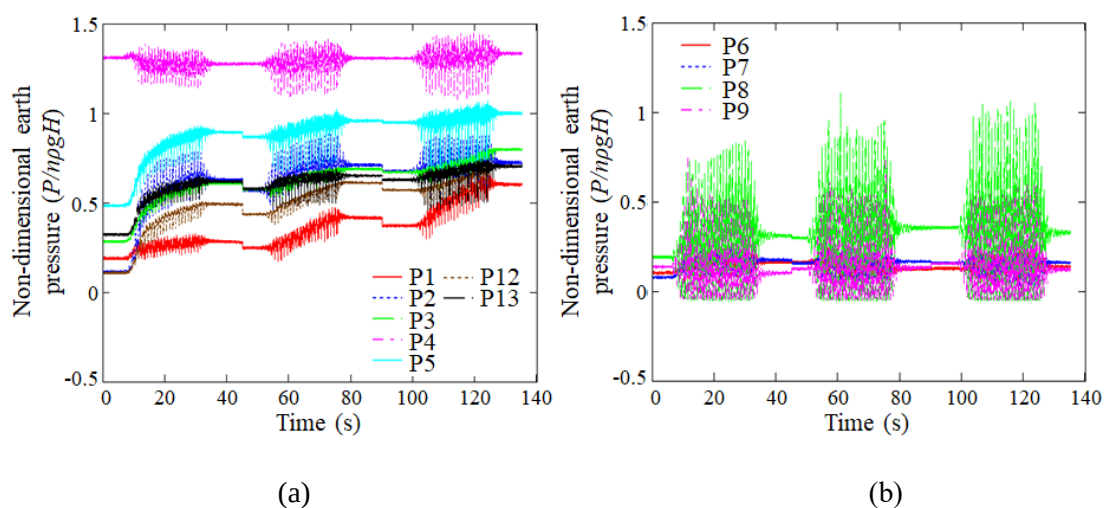
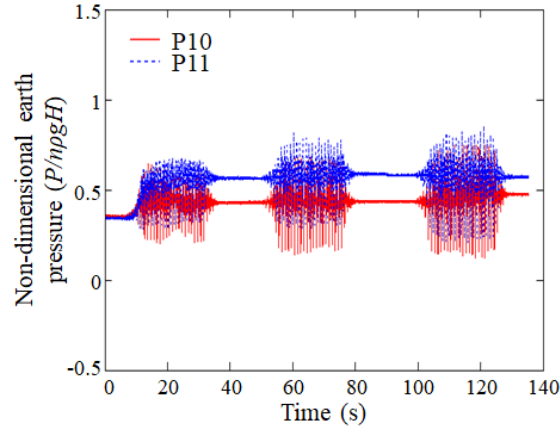


Fig. 9-18 Slope crests' vertical and horizontal displacement and slope surfaces' horizontal displacement with each shaking in Case 3

The symbols in **Fig. 9-18** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 3. During the first shaking, the value of LD5 went out of range, LD1 decreased and LD3 increased as the left target fell down.

Earth pressure





(c)

Fig. 9-19 Measured earth pressure with each shaking in Case 3: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Fig. 9-19 presents change of non-dimensional earth pressure with each shaking in Case 3, where the symbols can refer to those in **Fig. 9-9**.

Acceleration

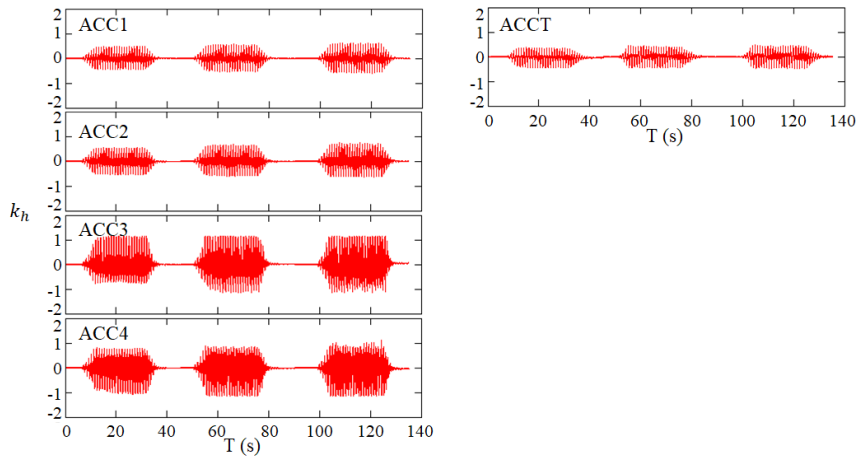


Fig. 9-20 Measured acceleration with each shaking in Case 3

Fig. 9-20 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

9.3.4 Case 4 (Number of shear pins: Left, 10, Right, 10; wooden pins, aligned in rows)

Sliding process

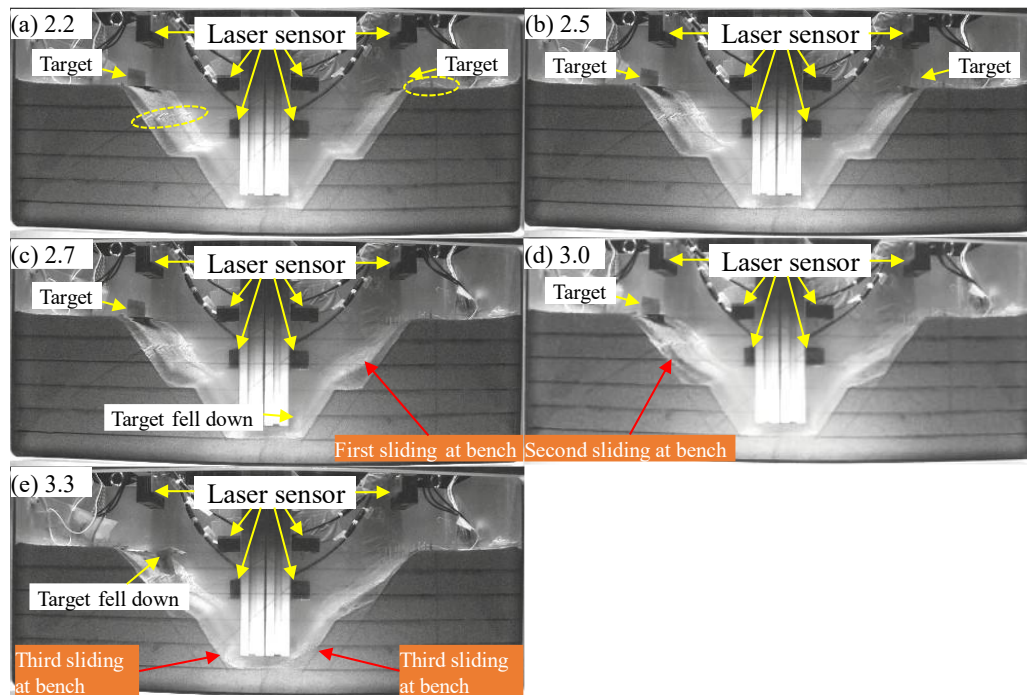


Fig. 9-21 Reaction of the model slopes to each shaking in Case 4: (a) stable state after the first shaking; (b) stable state after the second shaking; (c) first sliding at bench on the right slope; (d) second sliding at bench on the left slope; (e) third sliding at bench on both slopes

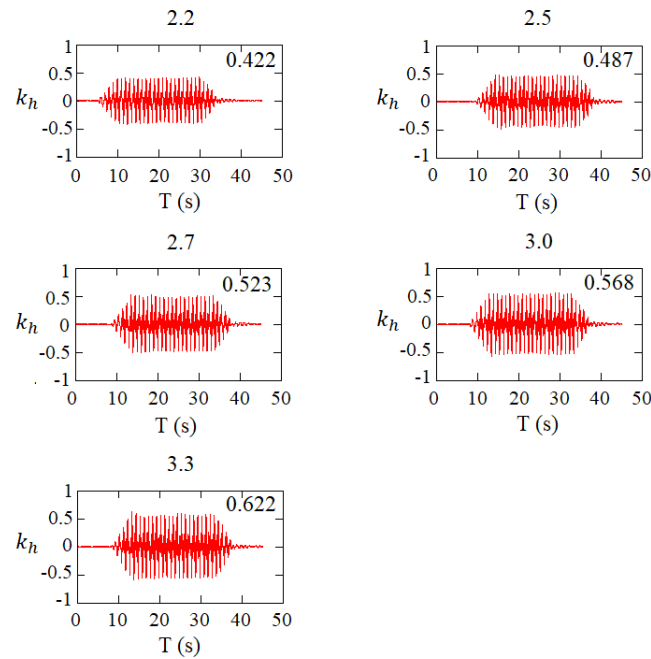


Fig. 9-22 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 4

In Case 4, a row of ten shear pins was installed on the left slope surface and on the right crest, as circled in **Fig. 9-21 (a)**. Both slopes remained stable after the first two shakings (see **Fig. 9-21 (a)** and **(b)**). During the third shaking with a magnification of 2.2 ($k_h=0.523$), the right second bench failed, and a surface sliding occurred on the left second bench (see **Fig. 9-21 (c)**). After the fourth shaking with a magnification of 3.0 ($k_h=0.568$), a sliding happened on the left second bench (see **Fig. 9-21 (d)**). The last shaking with a magnification of 3.0 ($k_h=0.622$) failed the first bench of both sides (see **Fig. 9-21 (e)**).

Displacement

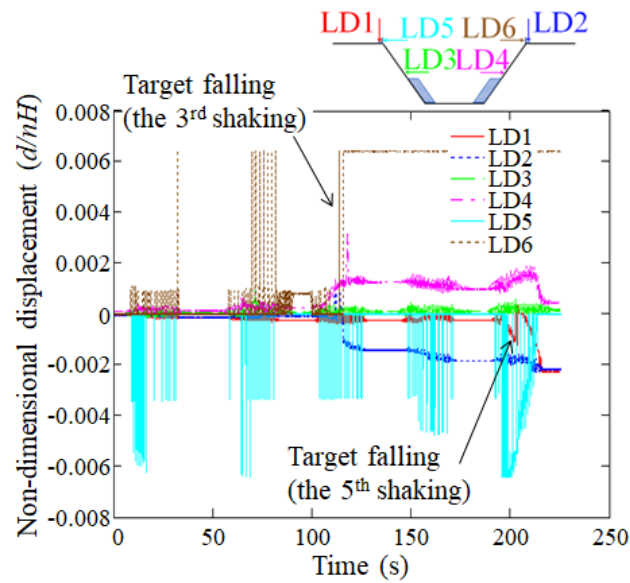


Fig. 9-23 Slope crests' vertical and horizontal displacement of and slope surfaces' horizontal displacement with each shaking in Case 4

The symbols in **Fig. 9-23** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 4. The right target fell down during the third shaking, causing the LD6 to go out of range, LD4 to increase, and LD2 to decrease.

Earth pressure

Fig. 9-24 presents change of non-dimensional earth pressure with each shaking in Case 4, where the symbols can refer to those in **Fig. 9-9**.

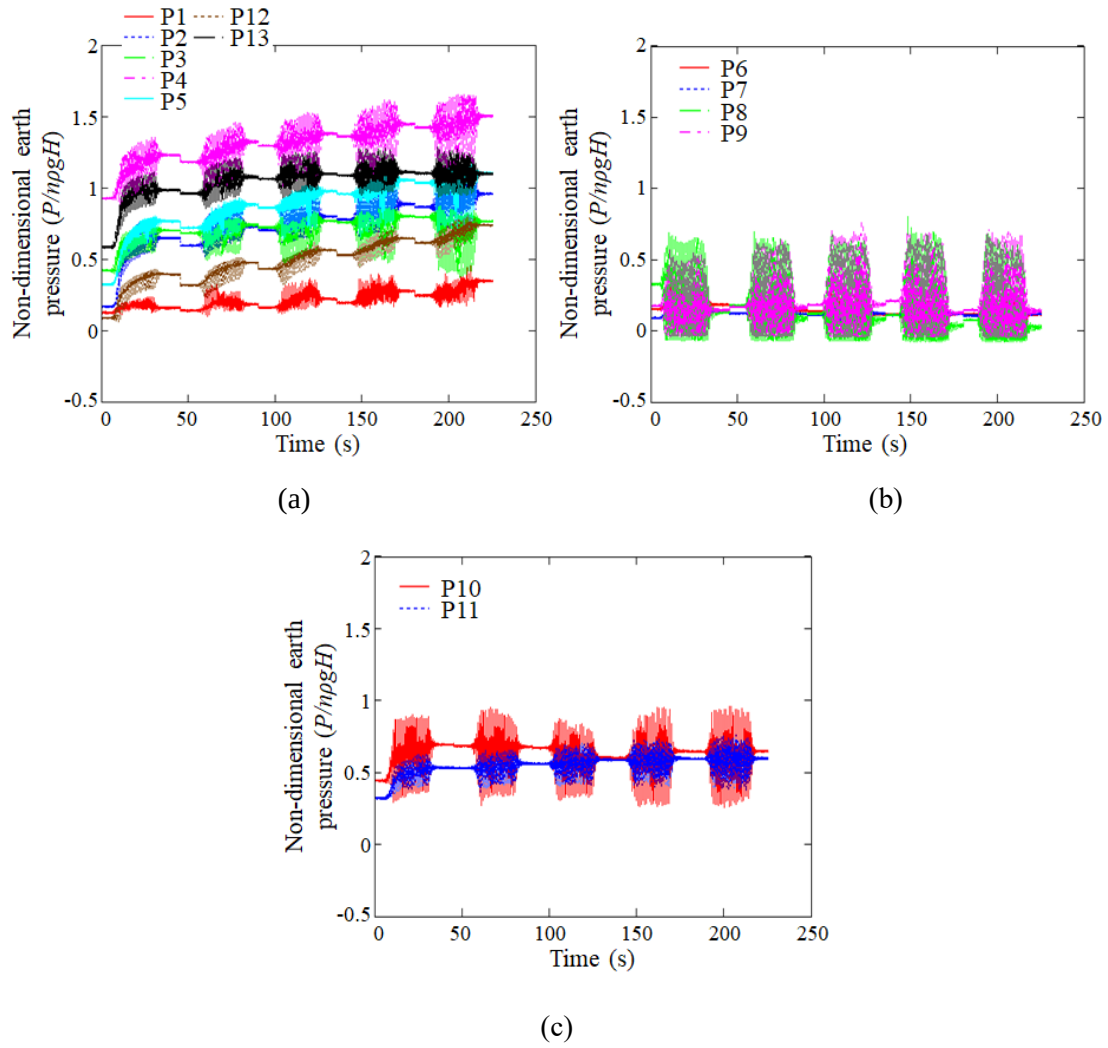


Fig. 9-24 Measured earth pressure with each shaking in Case 4: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

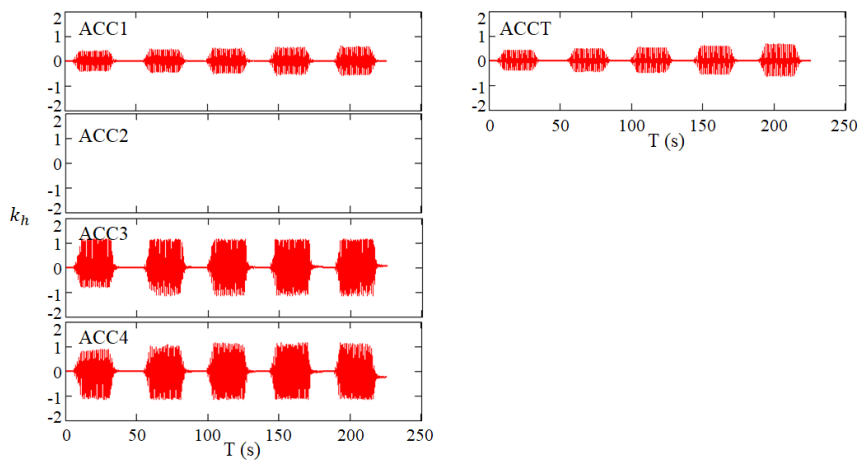


Fig. 9-25 Measured acceleration with each shaking in Case 4

Fig. 9-25 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations. ACC2 malfunctioned in this case.

9.3.5 Case 5 (Number of shear pins: Left, 19, Right, 10; wooden pins, aligned in rows)

Sliding process

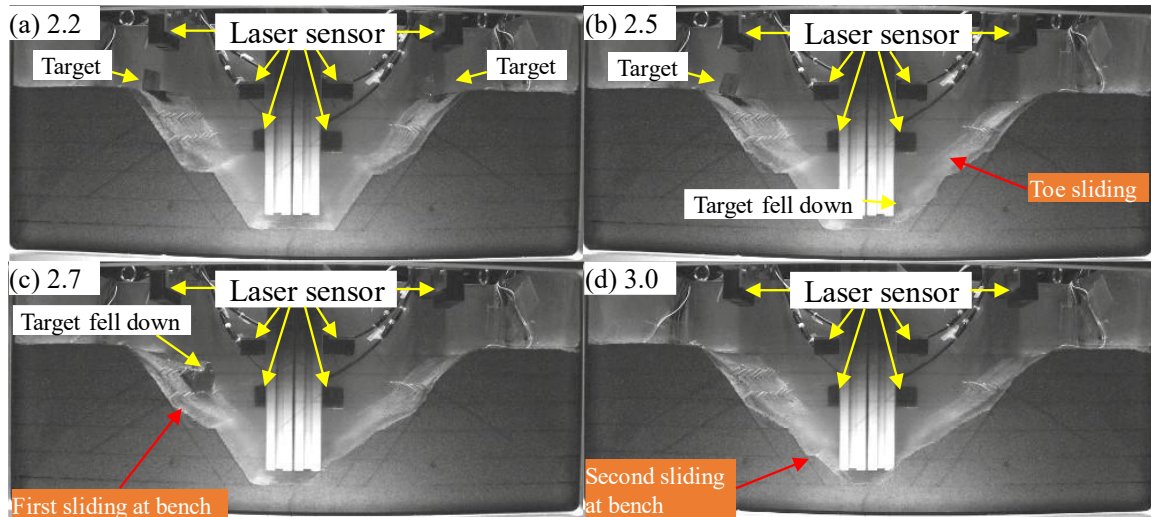


Fig. 9-26 Reaction of the model slopes to each shaking in Case 5: (a) surface sliding at the bench on the right slope; (b) toe sliding on the right slope; (c) first sliding at bench on the left slope; (d) second sliding at bench on the left slope

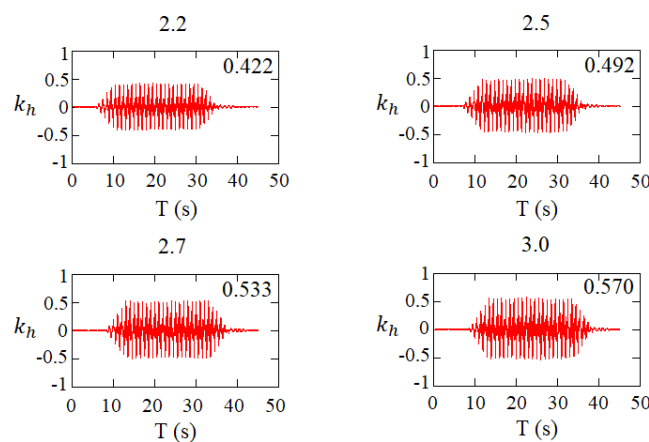


Fig. 9-27 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 5

In Case 5, two rows of nineteen shear pins were installed on the left slope surface, and one row of

ten pins was installed on the right slope surface. Some surface sliding happened on the right slope during the first shaking with a magnification of 2.2 ($k_h=0.422$) and the left slope remained stable (see **Fig. 9-26 (a)**). After the second shaking with a magnification of 2.5 ($k_h=0.492$), toe sliding happened on the right slope and some surface sliding was triggered on the left slope (see **Fig. 9-26 (b)**). Subsequently, a sliding happened on the left second bench after a shaking with a magnification of 2.7 ($k_h=0.533$) (see **Fig. 9-26 (c)**), followed by the failure of the first bench of the left slope during the last shaking with a magnification of 3.0 ($k_h=0.570$) (see **Fig. 9-26 (d)**).

Displacement

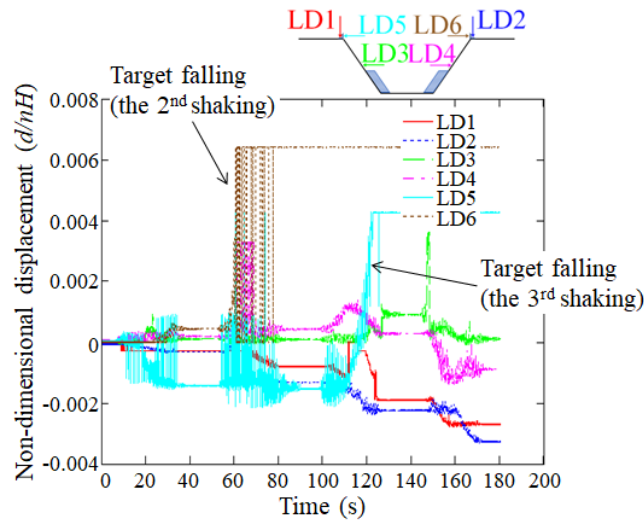


Fig. 9-28 Slope crests' vertical and horizontal displacement of and slope surfaces' horizontal displacement with each shaking in Case 5

The symbols in **Fig. 9-28** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 5. The right target fell down with the toe sliding during the second shaking, causing the LD6 to go out of range, LD4 to increase, and LD2 to decrease. During the third shaking, the left target fell down. LD5 went out of range, LD3 increased and LD1 decreased.

Earth pressure

Fig. 9-29 presents change of non-dimensional earth pressure with each shaking in Case 5, where the symbols can refer to those in **Fig. 9-9**.

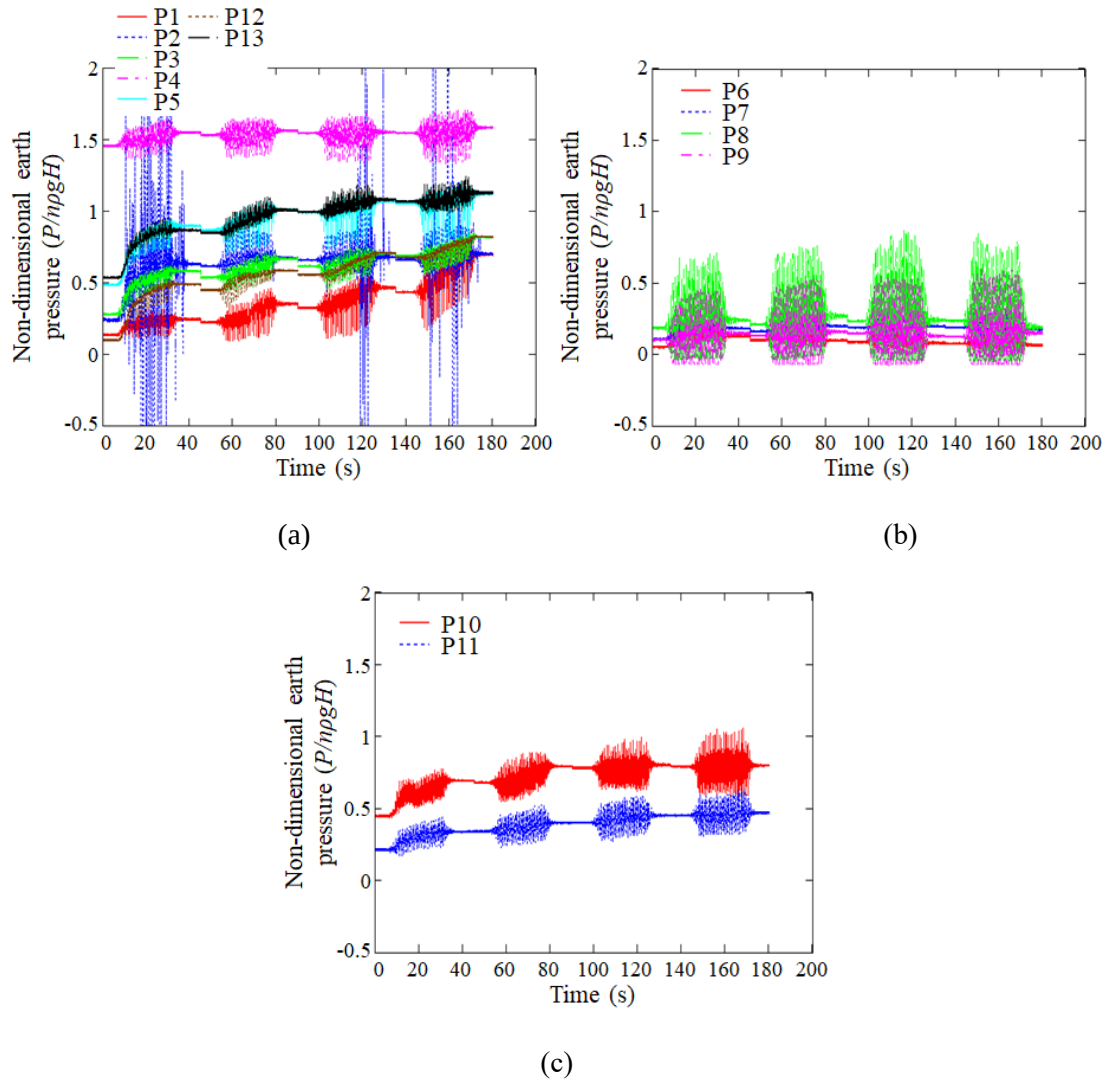


Fig. 9-29 Measured earth pressure with each shaking in Case 5: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 9-30 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations. ACC2 malfunctioned in this case.

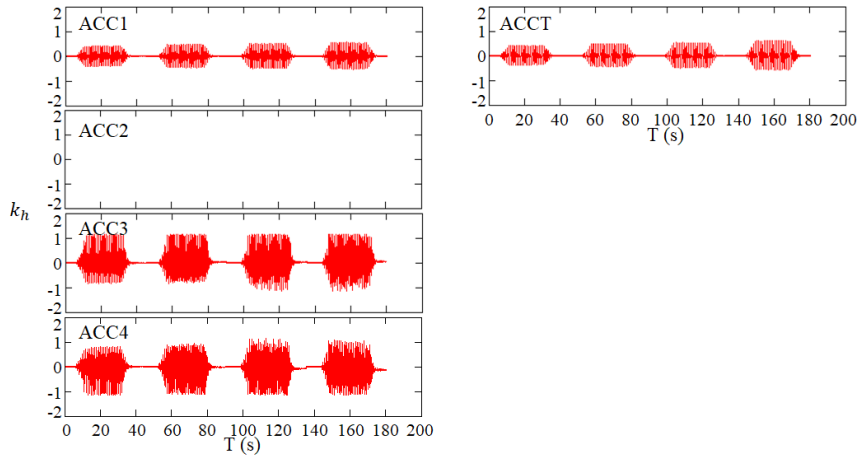


Fig. 9-30 Measured acceleration with each shaking in Case 5

9.3.6 Case 6 (Number of shear pins: Left, 10, Right, 19; wooden pins, aligned in columns)

Sliding process

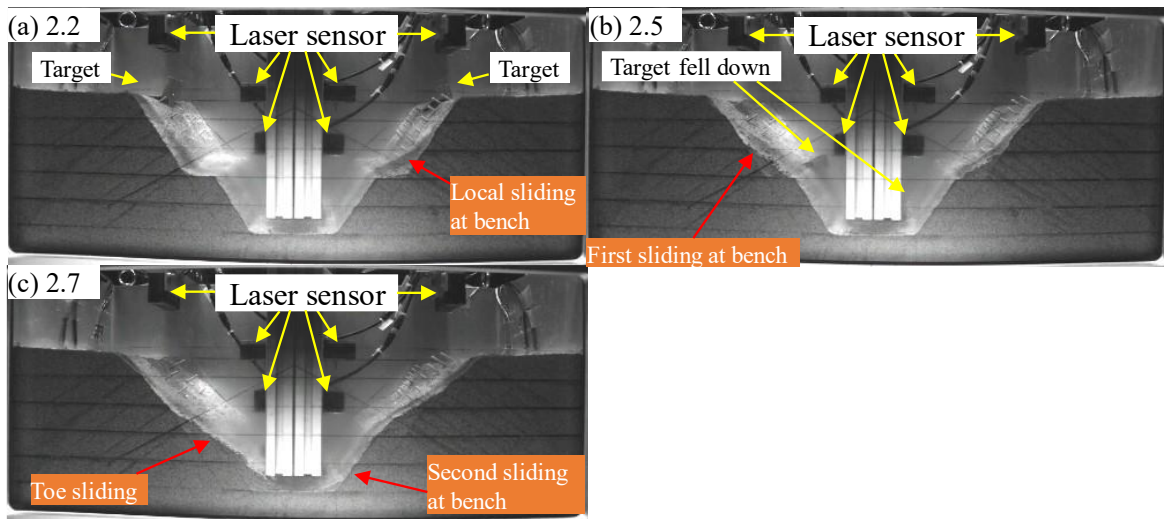


Fig. 9-31 Reaction of the model slopes to each shaking in Case 6: (a) local sliding at bench on the right slope; (b) first sliding at bench on the left slope; (c) toe sliding on the left slope and second sliding at bench on the right slope

In Case 6, two columns of ten and nineteen shear pins were installed on the left and right slope surfaces, respectively. A local sliding near the glass occurred on the right second bench during the first shaking with a magnification of 2.2 ($k_h=0.437$) (see **Fig. 9-31 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.519$), the left second bench failed (see **Fig. 9-31 (b)**). The last shaking with a magnification of 2.7 ($k_h=0.566$) triggered a toe sliding on the left slope and a sliding on the first bench on the right slope (see **Fig. 9-31 (c)**).

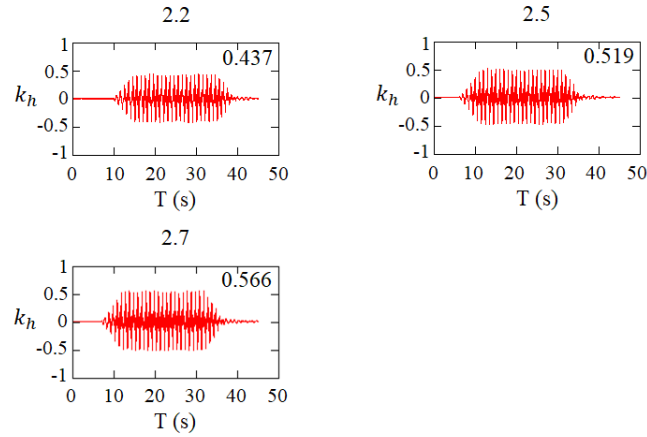


Fig. 9-32 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 6

Displacement

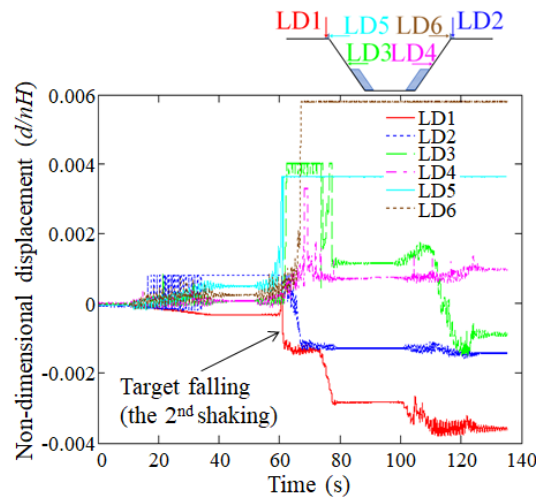


Fig. 9-33 Slope crests' vertical and horizontal displacement and slope surfaces' horizontal displacement with each shaking in Case 6

The symbols in **Fig. 9-33** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 6. During the second shaking, the targets on both crests fell down. LD5 and LD6 went out of range. LD1 and LD2 decreased. LD3 and LD4 increased.

Earth pressure

Fig. 9-34 presents change of non-dimensional earth pressure during each shaking in Case 6, where the symbols can refer to those in **Fig. 9-9**.

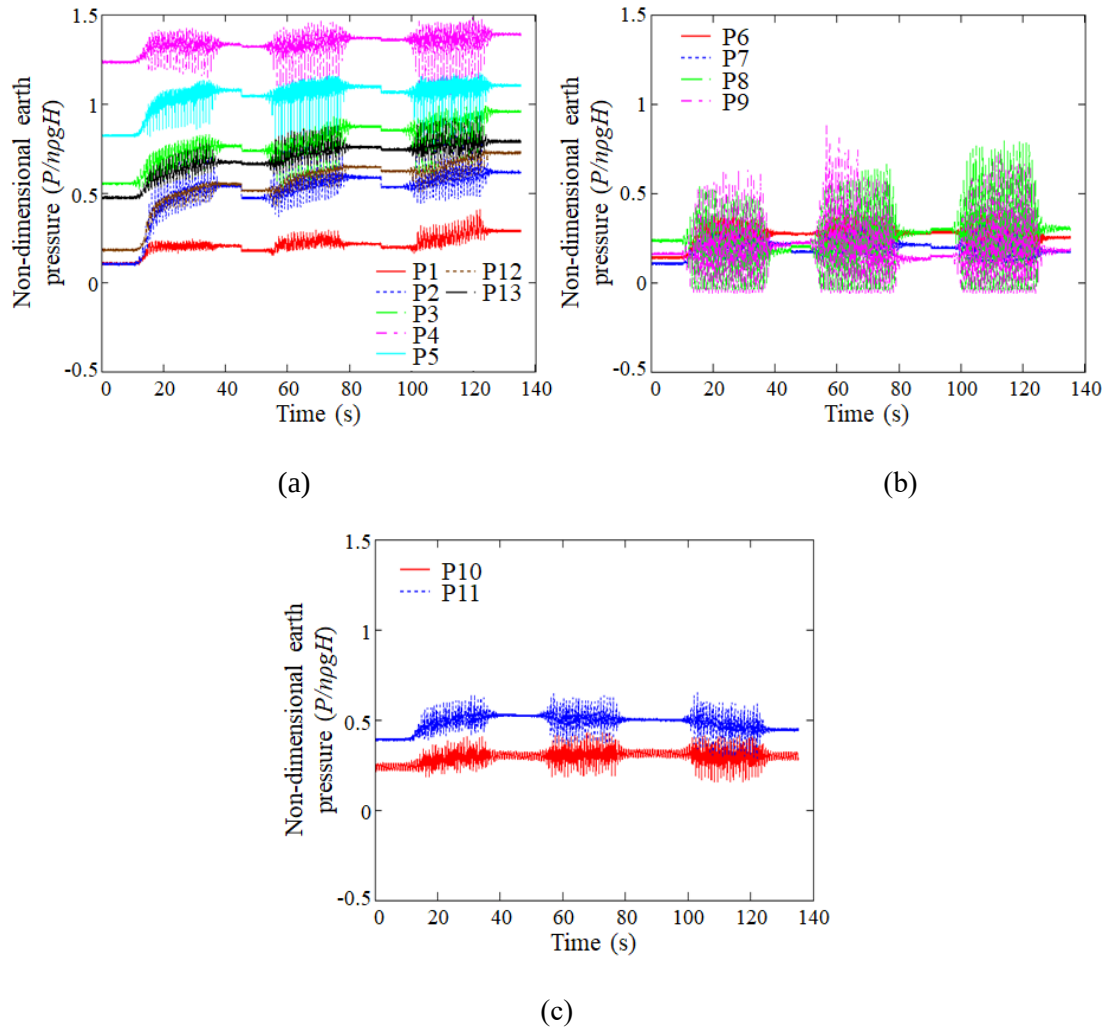


Fig. 9-34 Measured earth pressure with each shaking in Case 6: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

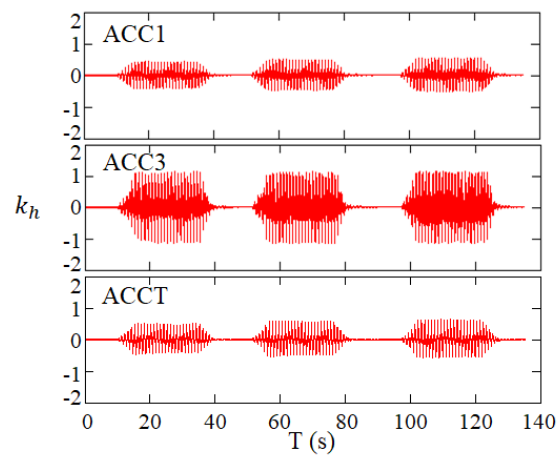


Fig. 9-35 Measured acceleration with each shaking in Case 6

Fig. 9-35 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

9.3.7 Case 7 (Number of shear pins: Left, 9, Right, 16; wooden pins, aligned in matrix)

Sliding process

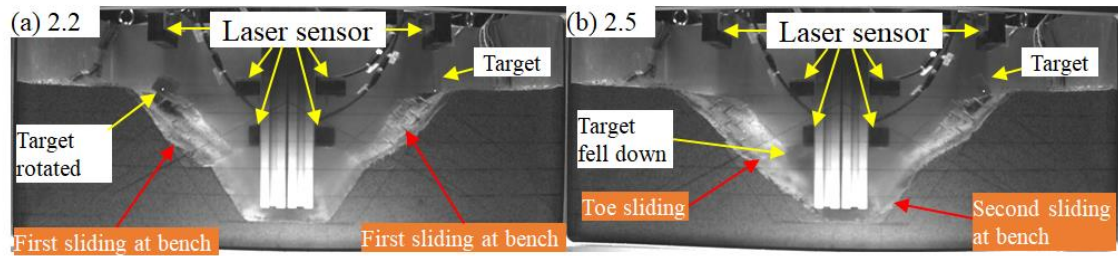


Fig. 9-36 Reaction of the model slopes to each shaking in Case 7: (a) first sliding at bench on both slopes; (b) toe sliding on the left slope and second sliding at bench on right slope

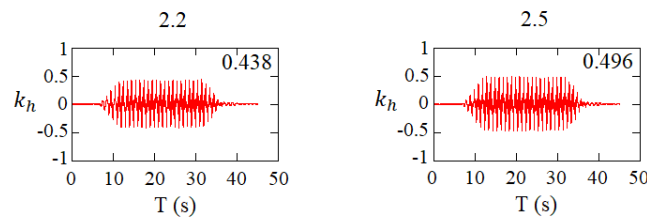


Fig. 9-37 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 7

In Case 7, shear pins arranged in a 3×3 and 4×4 matrix were inserted on left and right slope surfaces, respectively. The second bench of both sides failed at the first shaking with a magnification of 2.2 ($k_h=0.438$) (see **Fig. 9-36 (a)**). During the second shaking with a magnification of 2.5 ($k_h=0.496$), a toe sliding was induced on the left slope and a sliding happened at the first bench of the right slope (see **Fig. 9-36 (b)**).

Displacement

The symbols in **Fig. 9-38** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 7. The rotation of the left target caused by the first sliding at bench resulted in the increase of LD5 and decrease of LD1. During the second shaking, the left target fell down, causing the LD5 to go out of range and LD1 to decrease.

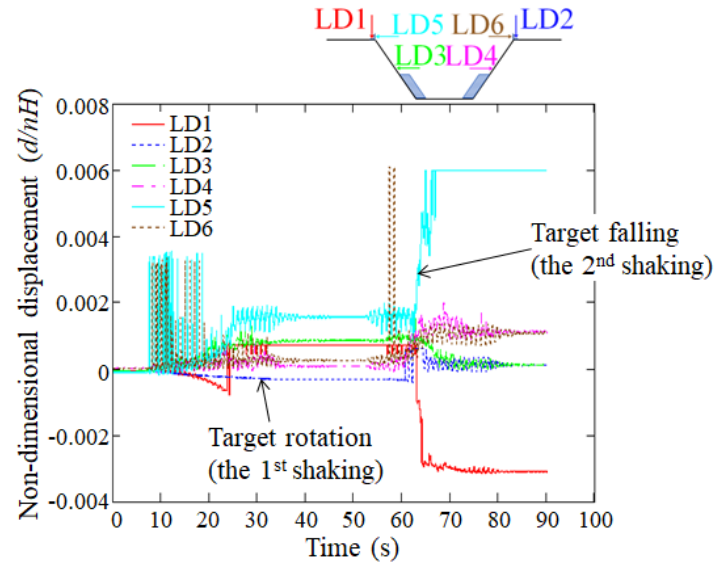
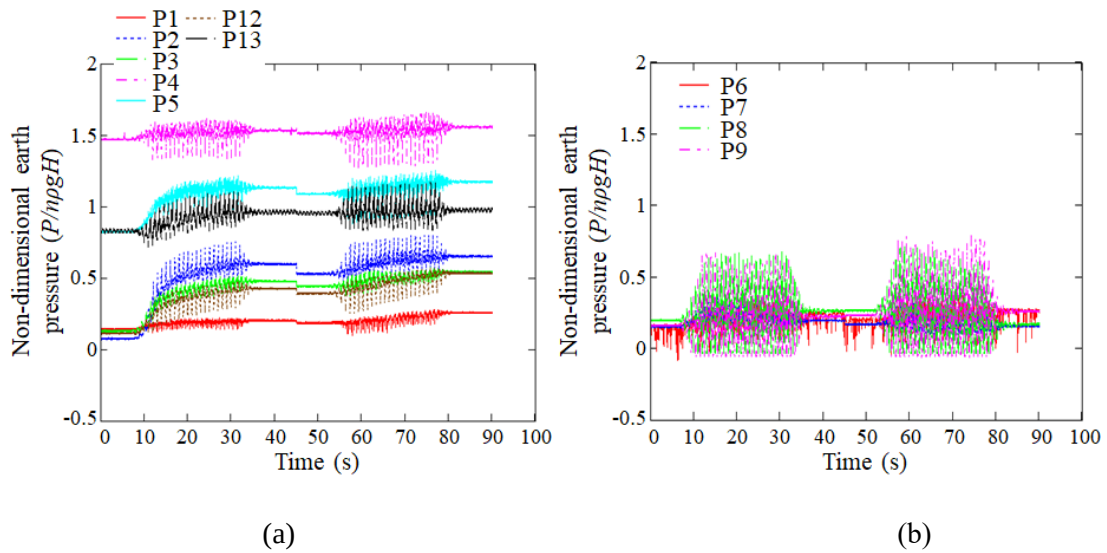
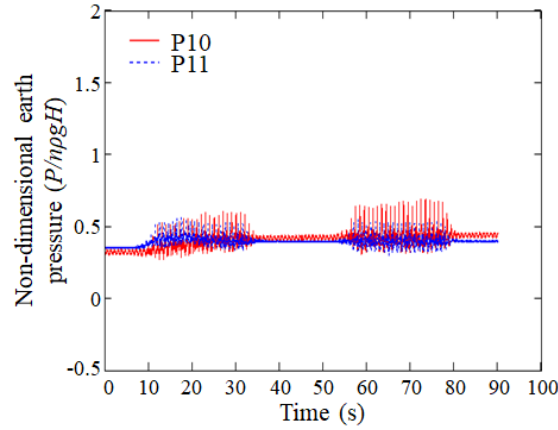


Fig. 9-38 Slope crests' vertical and horizontal displacement and slope surfaces' horizontal displacement with each shaking in Case 7

Earth pressure

Fig. 9-39 presents change of non-dimensional earth pressure during each shaking in Case 7, where the symbols can refer to those in **Fig. 9-9**.





(c)

Fig. 9-39 Measured earth pressure with each shaking in Case 7: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Acceleration

Fig. 9-40 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

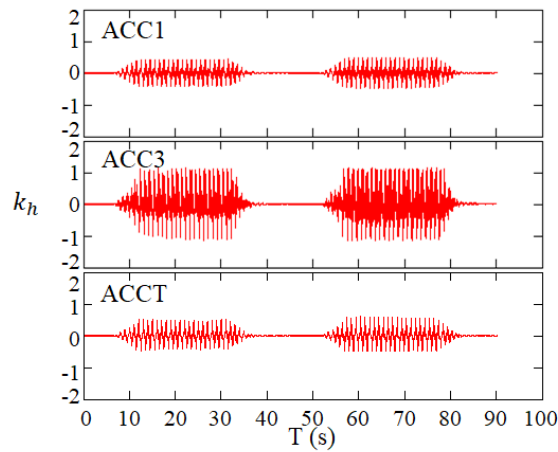


Fig. 9-40 Measured acceleration with each shaking in Case 7

9.3.8 Case 8 (Number of shear pins: Left, 8, Right, 14; wooden pins, aligned in matrix)

Sliding process

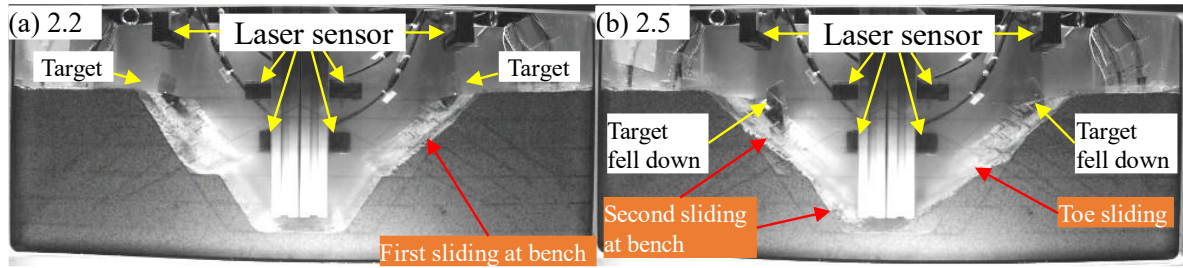


Fig. 9-41 Reaction of the model slopes to each shaking in Case 8: (a) first sliding at bench on the right slope; (b) second sliding at bench on the left slope and toe sliding of the right slope

In Case 8, gaps between pins were narrowed from Case 7. During the first shaking with a magnification of 2.2 ($k_h=0.447$), a sliding happened on the right second bench (see **Fig. 9-41 (a)**). After the second shaking, sliding occurred on the first and second bench separately on the left slope while a toe sliding was triggered on the right slope (see **Fig. 9-41 (b)**).

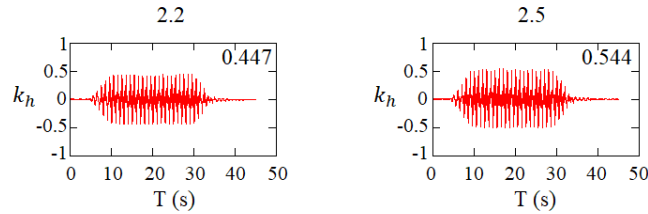


Fig. 9-42 Input waves in terms of horizontal seismic coefficient and their maximum absolute values corresponding to each magnification in Case 8

Displacement

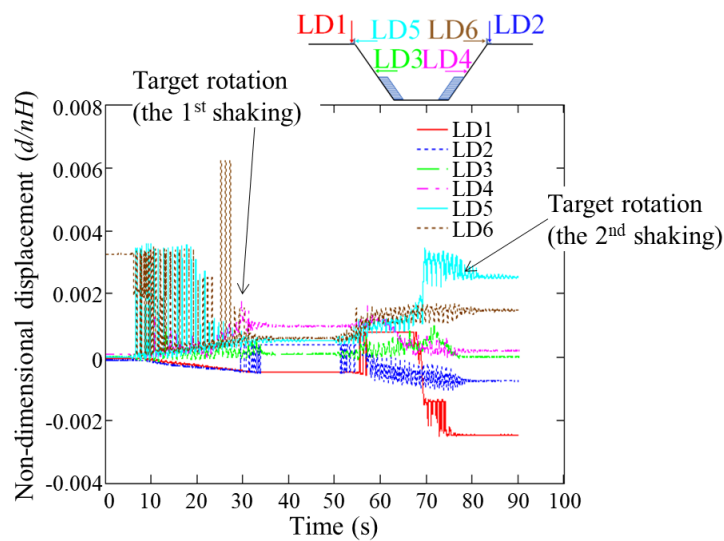


Fig. 9-43 Slope crests' vertical and horizontal displacement and slope surfaces' horizontal displacement with each shaking in Case 8

The symbols in **Fig. 9-43** have the same meanings as those in **Fig. 9-8**, illustrating the variation of non-dimensional displacement with respect to each shaking in Case 8. The change of values of laser sensors were caused by the rotation of targets in the two times of shaking.

Earth pressure

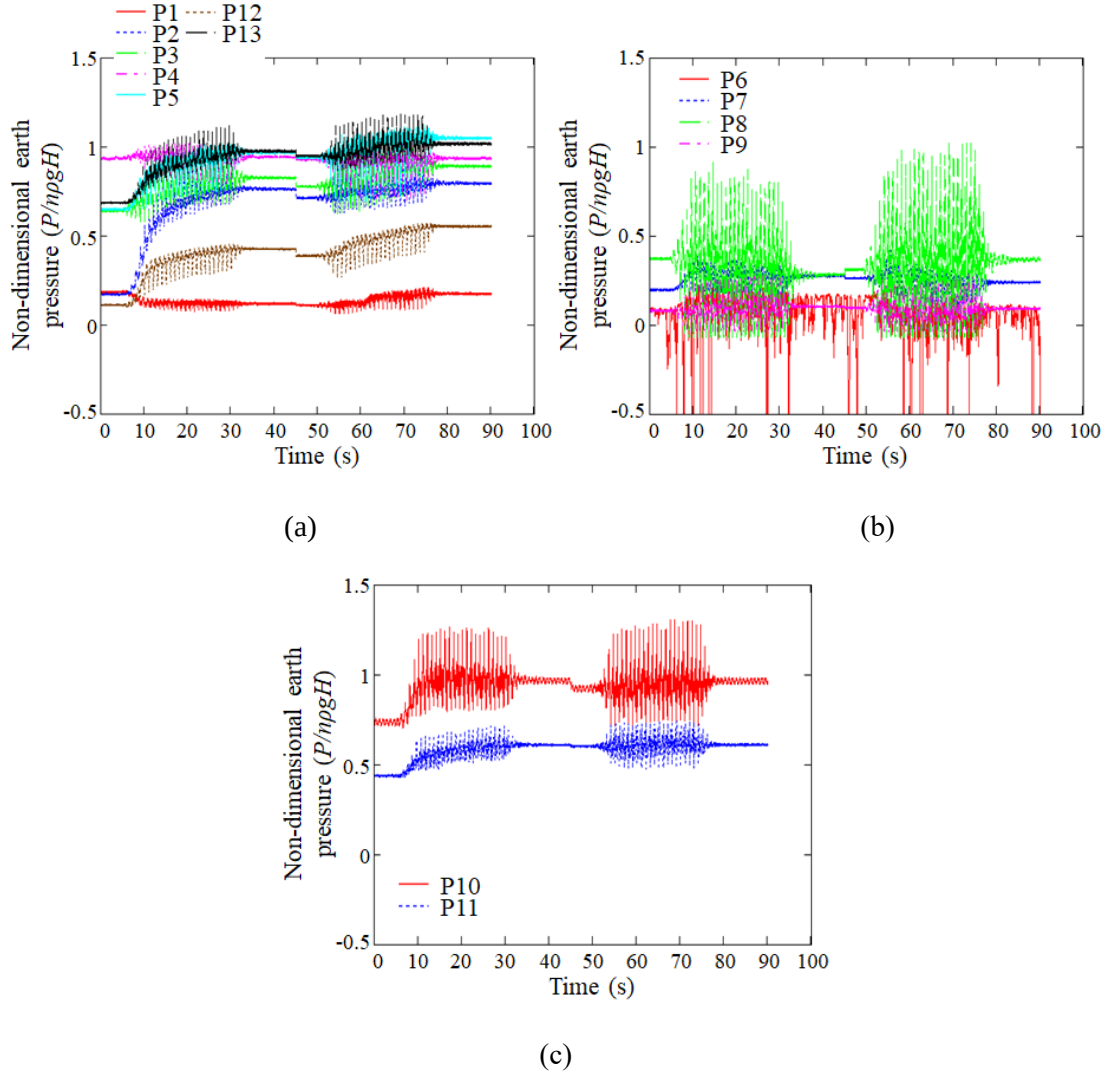


Fig. 9-44 Measured earth pressure with each shaking in Case 8: (a) bottom earth pressure; (b) horizontal earth pressure at half height; (c) vertical earth pressure at half height

Fig. 9-44 presents change of non-dimensional earth pressure during each shaking in Case 8, where the symbols can refer to those in **Fig. 9-9**.

Acceleration

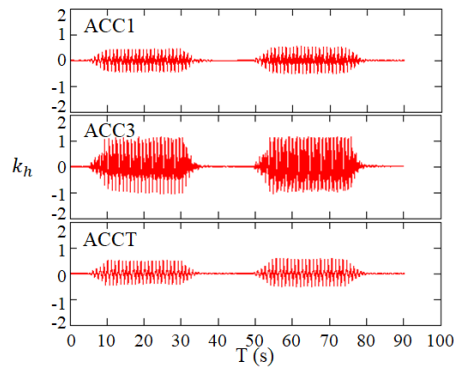


Fig. 9-45 Measured acceleration with each shaking in Case 8

Fig. 9-45 illustrates the measured accelerations at various locations of the model slope during each shaking. ACCT records the output wave of shaking table. The acceleration values have been converted to the prototype scale k_h by dividing 50g. The seismic coefficient phenomenon is evident, as the acceleration is greater at higher elevations.

9.4 Discussion

9.4.1 Effect of reinforcement with different alignments of shear pins

In general, during relatively low-intensity shaking, the upper bench of the slope could fail first, followed by the lower bench during more intense shaking. However, the whole slope could also fail at once sometimes. **Fig. 9-46** summarized the failure seismic coefficient according to different alignments of shear pins, where models A~D have been repeated once.

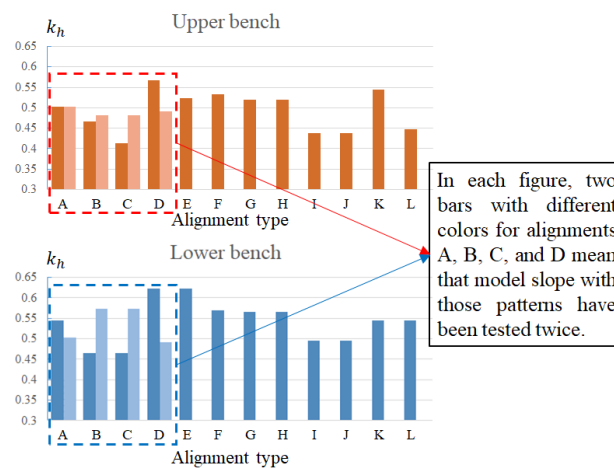


Fig. 9-46 Seismic coefficients at failure corresponding to various types of alignment

The first experiment with model D dominates all the patterns in the stability of both upper and lower benches, consisting of a row of ten wooden shear pins inserted on the middle of the upper bench

perpendicularly to the slope surface. This alignment was symmetric to the center line and the distance between the centers of each two pins was 1 m. Apart from this pattern, other models aligned in rows (E and F) also increased the slope stability. In addition, the sliding surface of those types were shallower than others. Particularly, when comparing the first test of models B and C, and that of models D and E, which belong to the same model slope, the upper bench with pins installed on the slope surface was more stable than with pins inserted on the slope crest. This may be because the sliding surface was quite shallow, and these vertically placed pins could not penetrate through it. Models with pins arranged in columns (G and H) also increased the slope stability to some extent. By contrast, model I and J with 3×3 and 4×4 arranged shear pins were likely to undermine the slope stability compared to the original model A, while model K and L with a narrowed gap between pins showed some increased stabilization. Thus, the distance between pins should be close enough, otherwise, it may backfire.

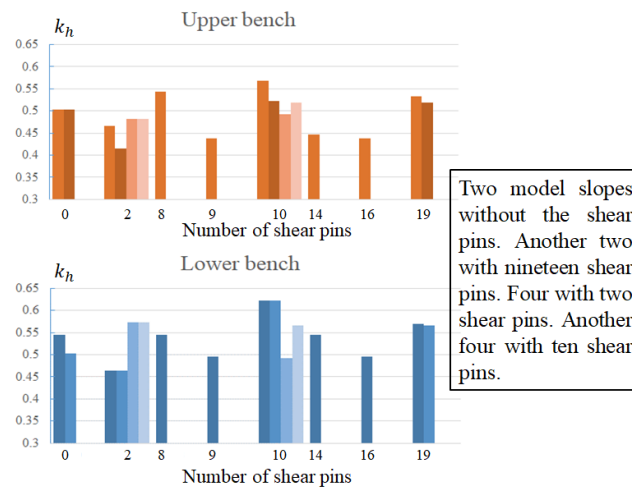


Fig. 9-47 Seismic coefficients at failure corresponding to various number of shear pins

When the failure seismic coefficient was compared based on the number of shear pins as shown in **Fig. 9-47**, no explicit trend can be observed; thus, the stability of the benched slope with shear pins does not depend on number of pins. Assumption that shear pins functioning through adding normal force on the sliding surface by its lateral resistance could not be validated in dynamic condition.

9.4.2 Overburden pressure at the bottom

Seven earth pressure sensors (P1 to P5, and P12 and P13) were placed along the longitudinal centerline at the model bottom to monitor the variation of overburden pressure during the ground motions. P1 was positioned under the center of the model. P2 and P13 were placed below the slope surface. P3 and P12 were located vertically below the slope toes, regardless the first bench. P4 and P5 were placed vertically below the slope crests. **Fig. 9-48** compares the normalized overburden height through back calculation, before and after seismic loading, with the normalized actual overburden height. The earth pressure increased after the shaking for all cases.

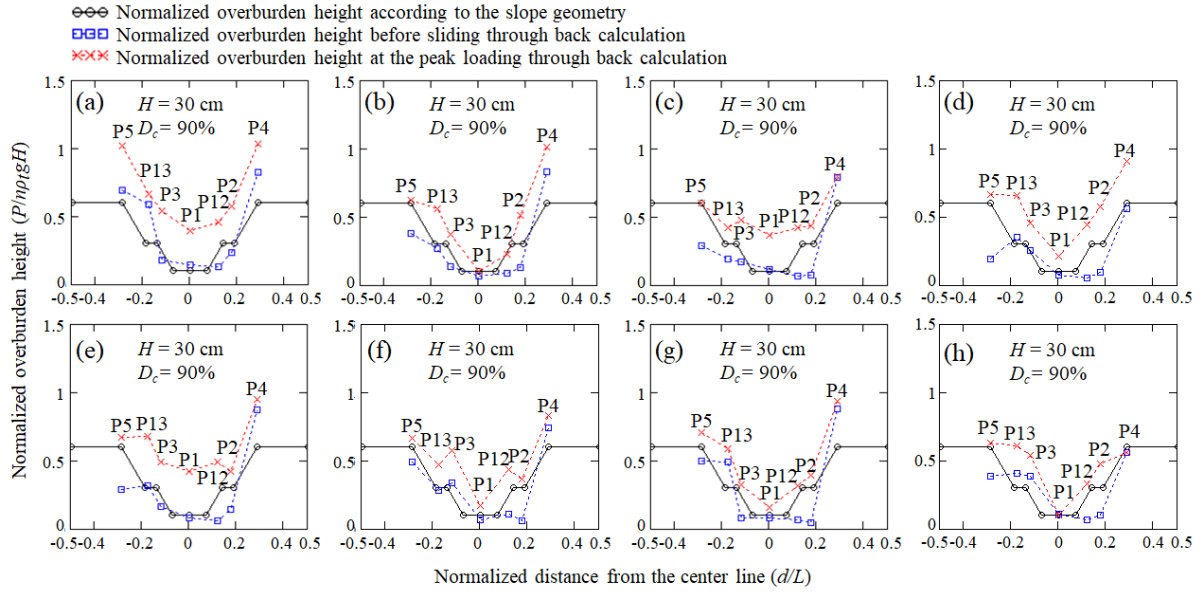
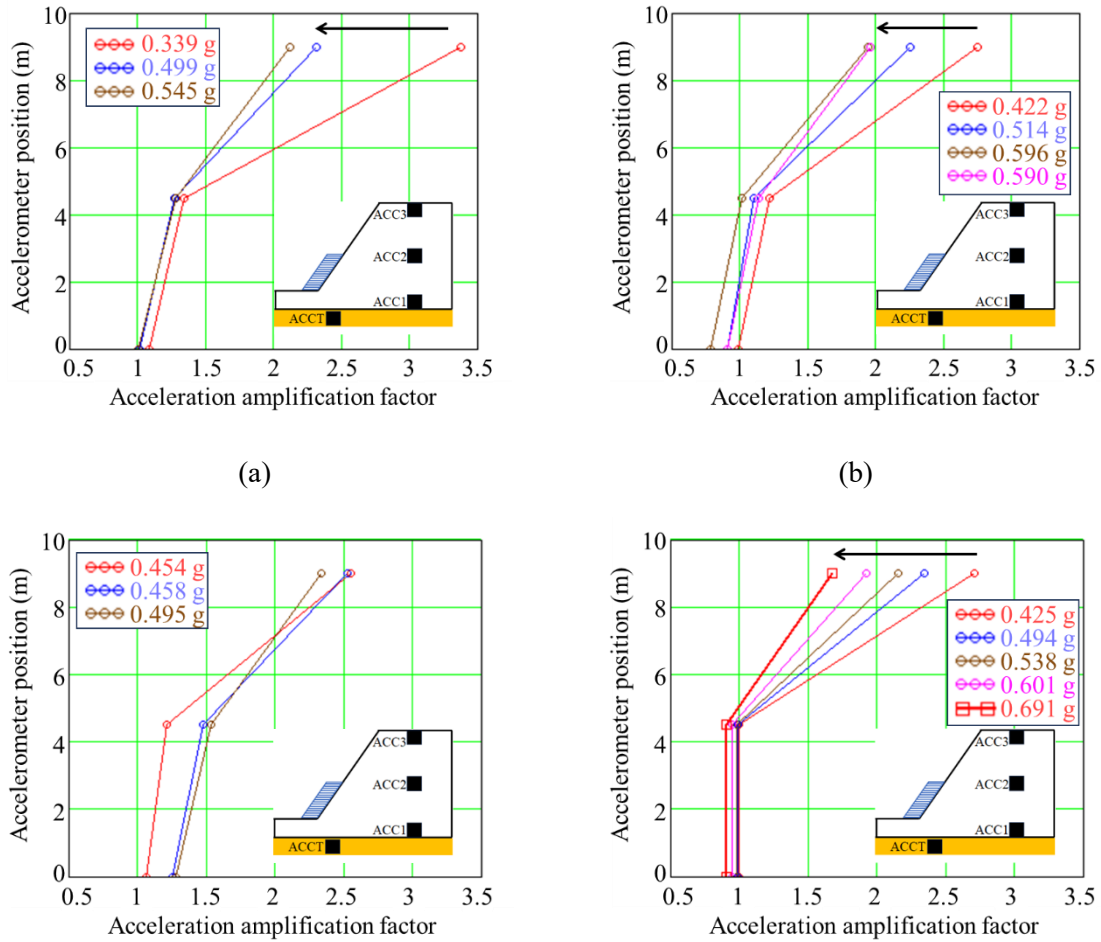


Fig. 9-48 Normalized overburden height at bottom of the model before and after seismic loading: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7; (h) Case 8

9.4.3 Seismic amplification with increasing magnitude of ground motions



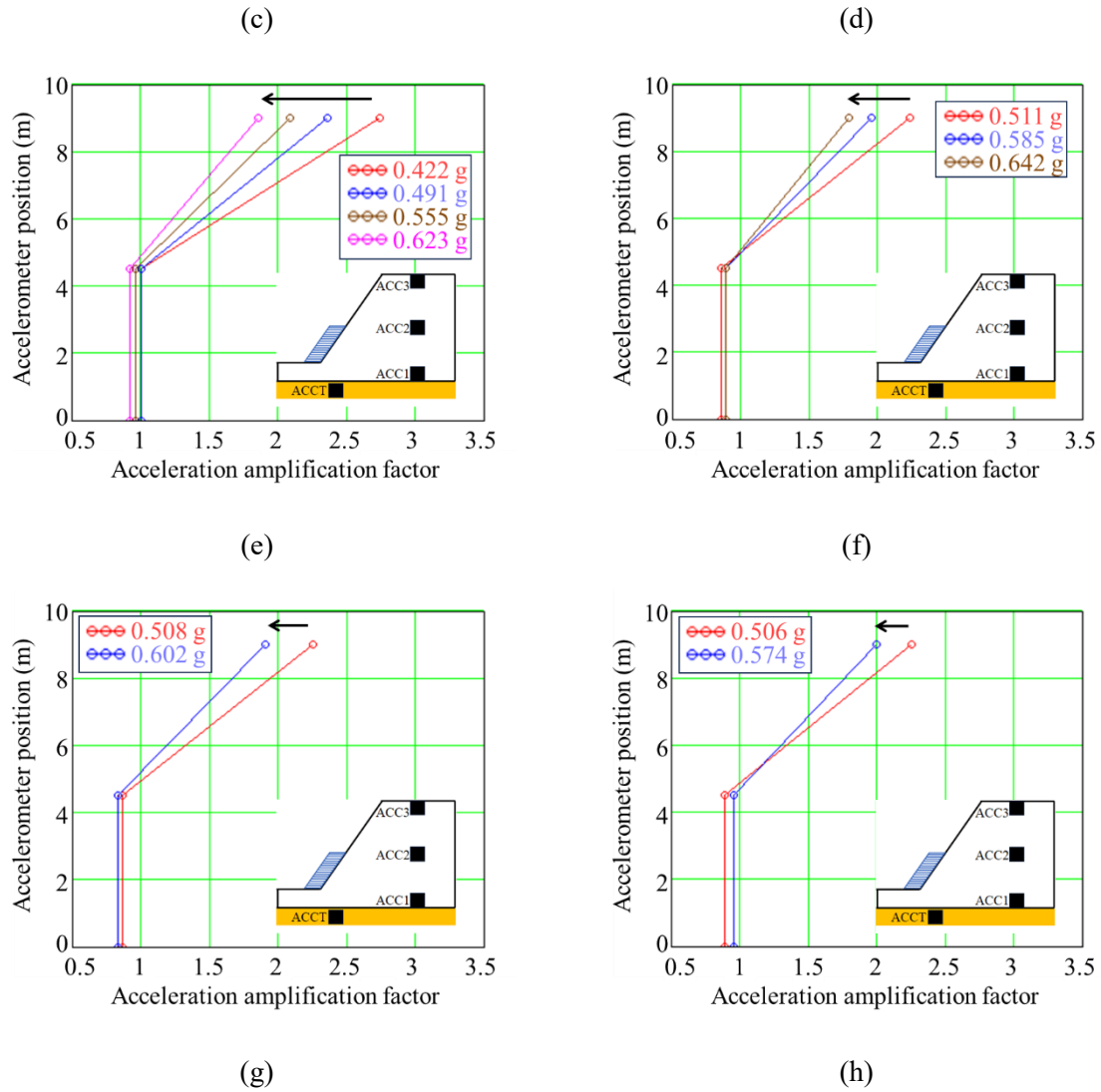


Fig. 9-49 Acceleration amplification factors of the benched slope retrofitted by shear pins: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7; (h) Case 8

The dimensionless acceleration amplification factor (AAF) has been explained in Chapter 6, which can depict the elevation amplification effective (EAE) in a slope subjected to ground motions. **Fig. 9-49** illustrates elevation amplification effect in all eight cases of the benched slopes retrofitted by shear pins. The ACC2 malfunctioned from Case 4 to Case 8, which is set the same value as ACC1. The acceleration increases nonlinearly with height. The level of amplification varies with magnitudes of input waves. The AAF on the top of model decreases with increasing magnitude of input wave.

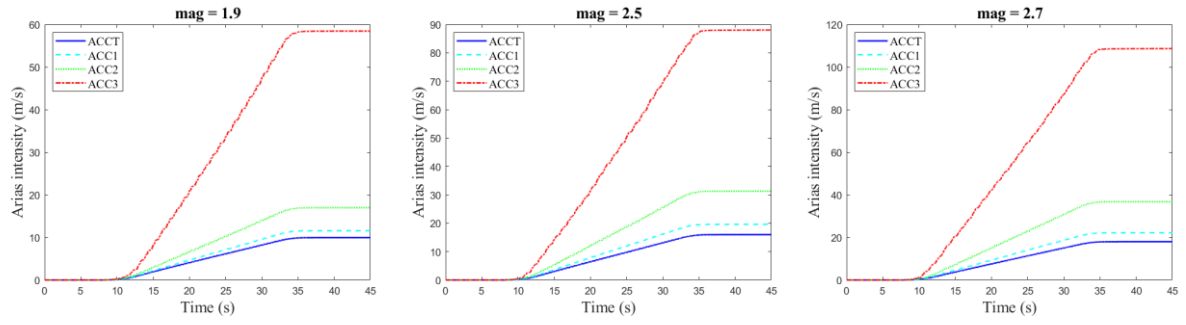


Fig. 9-50 Arias intensity evolution at various heights in Case 1

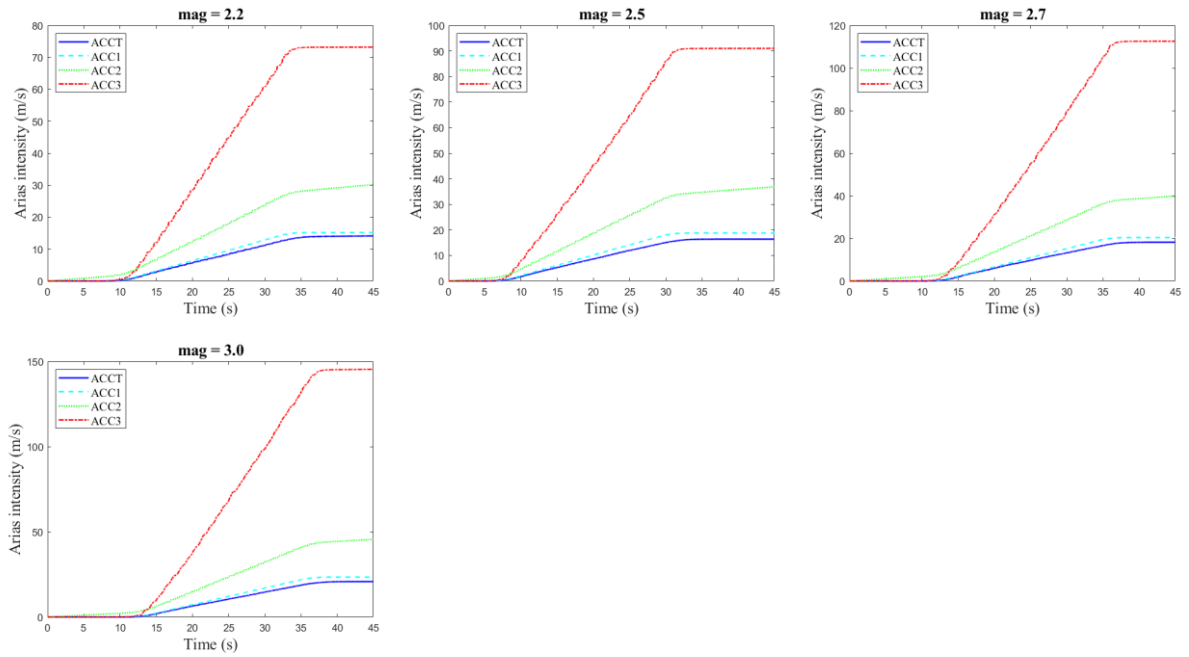


Fig. 9-51 Arias intensity evolution at various heights in Case 2

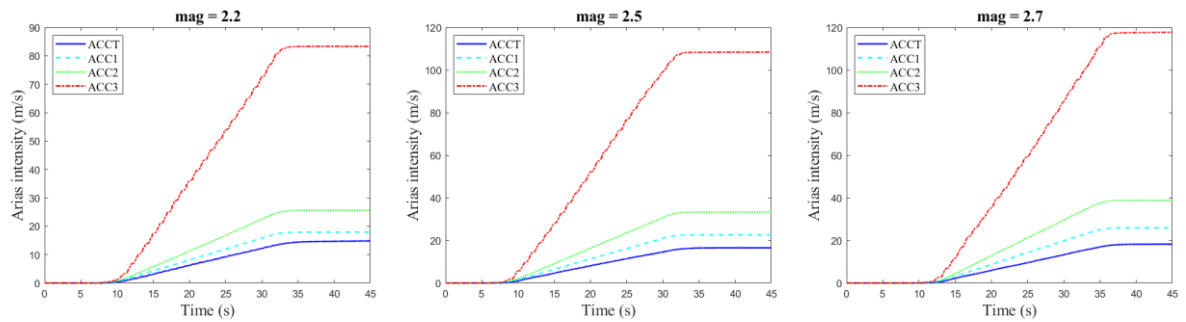


Fig. 9-52 Arias intensity evolution at various heights in Case 3

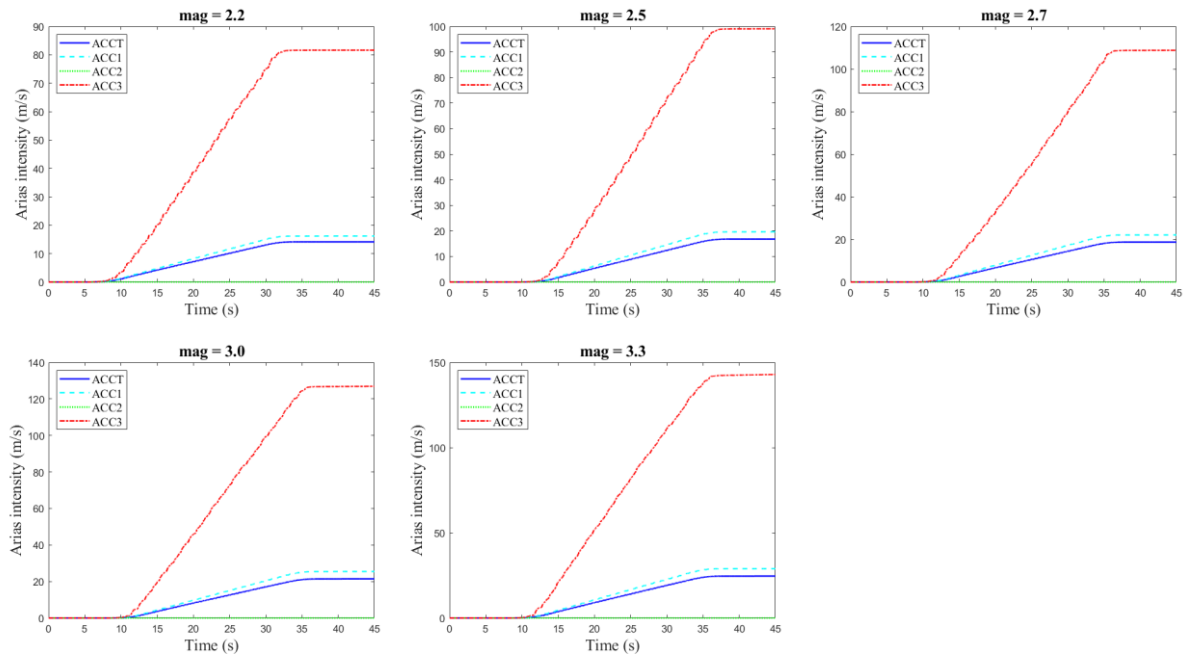


Fig. 9-53 Arias intensity evolution at various heights in Case 4

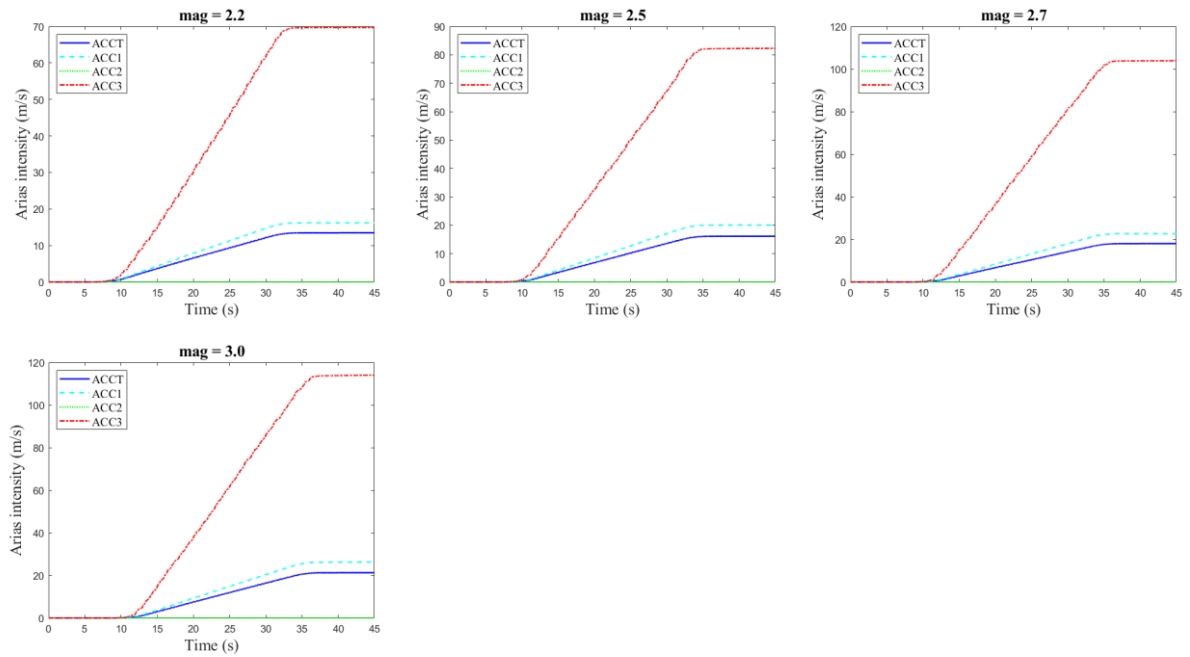


Fig. 9-54 Arias intensity evolution at various heights in Case 5

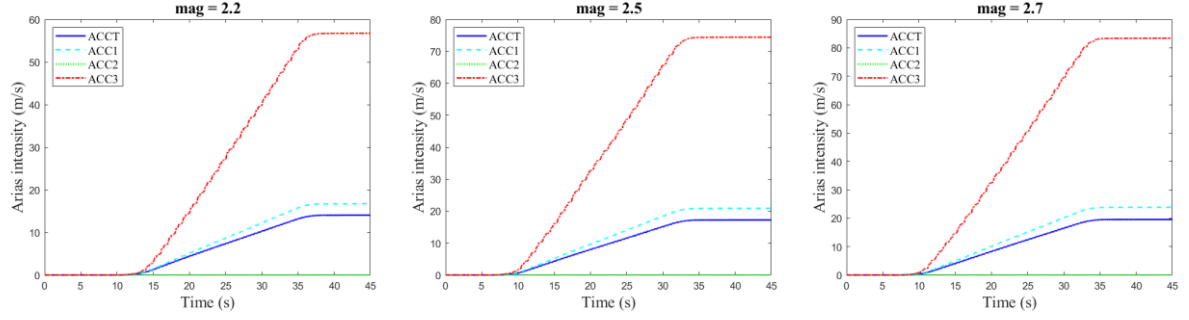


Fig. 9-55 Arias intensity evolution at various heights in Case 6

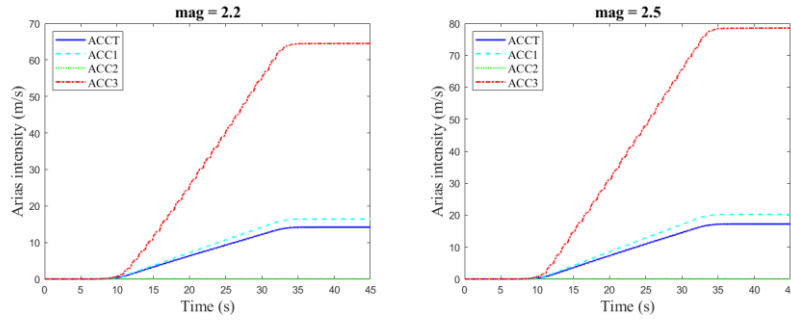


Fig. 9-56 Arias intensity evolution at various heights in Case 7

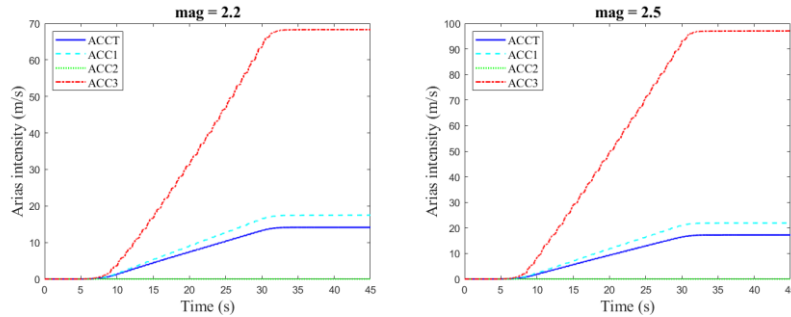
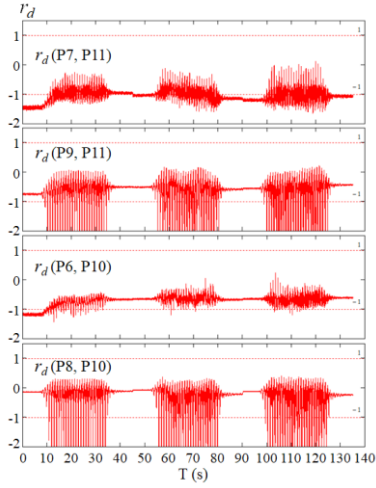


Fig. 9-57 Arias intensity evolution at various heights in Case 8

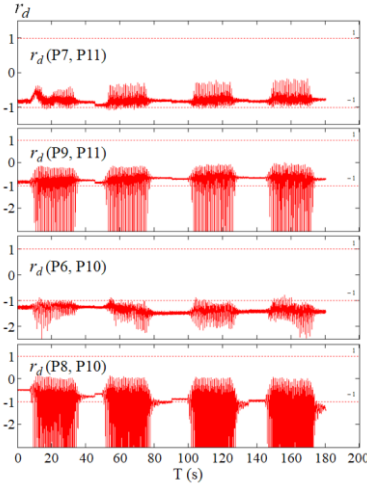
Fig. 9-50 to Fig. 9-57 depict the evolution of Arias intensity at different height of the model with increasing shaking intensity. From Case 4, ACC2 malfunctioned, its Arias intensity becomes zero.

9.4.4 Variation of the slope state with ground motions

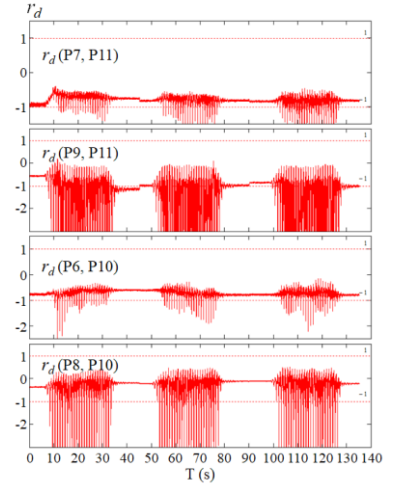
As mentioned in **section**, the reduction ratio, r_d , expressed by **Eq.** can represent the slope state, ranging from the active state to a passive state. The model slopes deformed under the centrifugal loading during the test, leading to the varied states. For each slope, two r_d values can be calculated, resulting in four r_d . For example, in the right slope of Case 1, a horizontal earth pressure sensor (P10 on the center line of the model) and two vertical earth pressure sensors (P6 on the back wall, P8 on the right wall) are located 9 cm above the bottom of the soil chamber, by combining P10 with P6 and P8 obtains two r_d (Tao & Pipatpongsa, 2023).



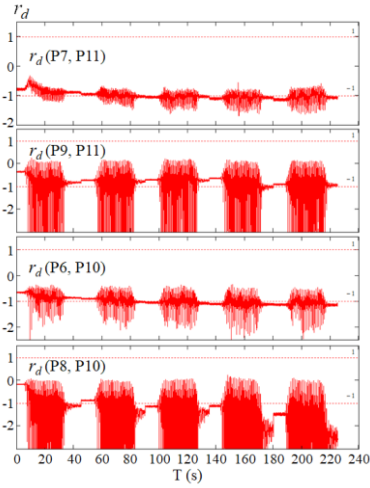
(a)



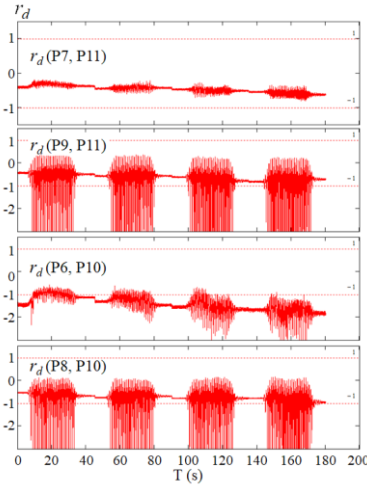
(b)



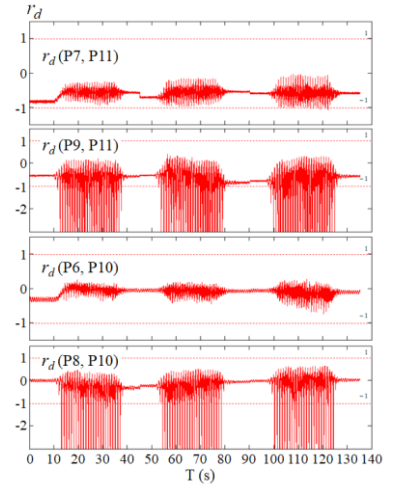
(c)



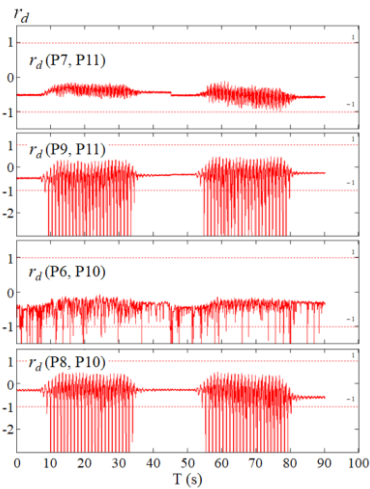
(d)



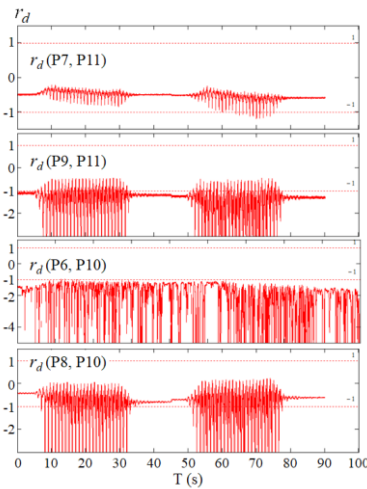
(e)



(f)



(g)



(h)

Fig. 9-58 Variation of reduction ratio with ground motions: (a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7; (h) Case 8

The variation of reduction ratio with each shaking is shown in **Fig. 9-58**. When the model reached 50G and was before shaking, it already achieved a limit active state with r_d being around -1. The first shaking with a low intensity always increased the value of r_d , and it can decrease after a stronger seismic loading. Moreover, r_d calculated by pressure sensors on the side wall (P8 and P9) were more pronounced than that by sensors on the left and right walls (P6 and P7). This means the slope part near the slope surface experienced more state variation during shaking.

9.5 Conclusions

This chapter evaluated the impact of shear pin alignments on benched slopes' dynamic stability. The installation of shear pins can enhance seismic resistance and alter the failure modes of benched slopes, providing insights for practical design applications. The main conclusions are listed as follows.

(1) Because of seismic amplification, the slope crest is the most vulnerable area and requires retrofitting.

(2) The presence of shear pins did not alter the acceleration amplification factor in the interior slope.

(3) Shear pins installed in rows with narrow spacing on the surface of the upper bench can enhance the dynamic stability of the benched slope. Column alignment with small gaps can also improve stability to some extent, but it may still induce local sliding.

(4) Alignment by matrix with a large gap between pins may weaken the slope integrity and reduce its stability. Therefore, the alignment of shear pins is more crucial than their quantity because the pins primarily bear shear force rather than tensile force under dynamic conditions.

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Chapter 10 Conclusions and future studies

This research focuses on benched slopes' stability with varying geometric configurations and loading modes. A comprehensive approach, combining centrifugal modeling and DEM simulation, was adopted to investigate the failure mechanisms of benched slopes. From these insights, simplified design methods were developed, and a feasible retrofitting technique using shear pins was proposed. It was found that: (1) Geometrical retrofitting is beneficial to slope. Adding a bench at the slope toe can enhance both the static and dynamic slope stability. This is because the bench provides additional resisting force to the slope toe, which experiences the most significant stress concentration. Generally, the retrofitting effect increases with bench width and height. However, this effect has limitations. When the bench height becomes excessively large, the overall slope stability decreases, and when the bench width exceeds a certain threshold, further increases in width do not improve stability. (2) The geometry of a slope significantly influences its failure modes. Larger bench dimensions can divide the slope into isolated sections, shifting the dominant sliding mode from toe sliding to bench sliding. This separation becomes particularly pronounced in benched slopes subjected to seismic forces, as seismic amplification renders the upper bench more vulnerable to failure.

More detailed conclusions obtained in each chapter are presented below.

In **Chapter 2**, a thorough review on benched slopes was conducted to establish the framework for this study. Previous research has provided invaluable insights; however, several research gaps were identified that must be addressed to meet engineering project needs. Subsequently, foundational methodologies were introduced to achieve the following research objectives:

- (1) Identify potential failure modes, critical height, and seismic resistance of benched slopes with various configurations and loading conditions through centrifugal model tests.
- (2) Propose a closed-form solution for calculating the *FS* of steep uniform slopes and extend it to three-dimensional and dynamic conditions for benched slopes. Utilize measured data from centrifugal tests to explore how to incorporate lateral resistance into the three-dimensional theoretical solutions.
- (3) Construct a 3D DEM model to validate the experimental results and identify additional influencing factors. Uncover details not easily observed in physical tests for benched slopes, such as full-field stress and deformation distributions, as well as particle-level force chain networks.
- (4) Based on the identified fragile areas of benched slopes under ground motions, develop a feasible seismic retrofit method using shear pins. Compare different shear pin alignments in dynamic centrifugal tests to determine the most effective configuration.

A series of studies were conducted to achieve these goals, as described below.

In **Chapter 3**, fundamental laboratory tests essential for the subsequent research were presented. The shear strength of Hiroshima sand, used in the experiments, was significantly influenced by compaction degree and water content, with higher strength observed at greater compaction levels. Under the same compaction degree, water content primarily affected the sand's cohesion rather than its internal friction angle.

In **Chapter 4**, the static stability of a laterally confined uniform steep slope was investigated, using a closed-form solution newly derived from 2D Culmann's method, which avoids the iterative process typically required. By incorporating Rankine's theory, this approach was extended to 3D conditions to consider lateral resistance. Static centrifugal model tests were conducted to validate these theories. Earth pressure sensors indicated the transition from passive to active states under increased loading, with the slope eventually reaching a limit active state, confirming the effectiveness of 2D theory for static analysis. Finally, predictions for the critical height using parameters obtained from direct shear tests aligned well with experimental results, validating the effectiveness of the proposed methods, particularly when adjusted for water content variability.

In **Chapter 5**, centrifugal model tests confirmed that benched slopes exhibit enhanced stability compared to uniform slopes, with failure modes influenced by bench geometries. Maintaining the overall slope inclination while adding a first bench at the slope toe can improve stability. As the first bench's height and width increase, the failure mode shifts from toe sliding to bench sliding, indicating an optimal dimension for the first bench based on failure mode transitions. When benched slopes are constructed with a fixed excavation volume, they may be less stable than uniform slopes, as the higher inclination of individual benches relative to the overall slope induces greater stress concentration. Additionally, toe sliding is likely when the height of the excavated section exceeds its width, whereas bench sliding tends to dominate when the width is greater. Therefore, to achieve a greater critical height for the entire slope, the former design is recommended, while the latter design is preferable for enhancing stability at the slope crest. When considering the first bench as a counterweight for the toe in theoretical predictions, it is essential to account for both the overburden and berm components.

In **Chapter 6**, the seismic stability of uniform steep slopes was analyzed using pseudo-static methods and dynamic centrifugal model tests, with a focus on shallow sliding triggered by varying ground motion intensities. The pseudo-static method effectively calculated the critical seismic coefficient considering a confidence interval for shear strength. During shaking, the reduction factor fluctuated, at times reaching passive states, suggesting that lateral resistance contributes to dynamic slope stability. Additionally, with installed accelerometers, the elevation amplification of the seismic wave can be observed. By using acceleration amplification factor (AAF) and Arias intensity, it was noticed that the reaction acceleration in the slope increased non-linearly with the height and became pronounced in the slope crest.

In **Chapter 7**, the dynamic stability of benched slopes was investigated through dynamic centrifugal tests and pseudo-static methods, considering various failure modes under 2D and 3D conditions. Lateral shear resistance proved essential for accurate predictions, as slopes may reach static or passive states during ground motions. The addition of a bench at the slope toe was found to enhance stability; however, the second bench generally exhibited lower stability due to seismic amplification. Failure modes shifted from toe sliding in uniform slopes to bench sliding in benched configurations. Higher benches were beneficial, and increasing the number of bench steps further enhanced seismic resistance, though the effect of increased bench width was limited.

In **Chapter 8**, 3D DEM simulations were conducted to understand the stability of benched slopes under static and dynamic conditions. DEM simulations can achieve good consistency with experimental results by applying the principles of centrifugal modeling, reducing the model size, and increasing the gravitational force. This approach maintains a similar stress level in the simulation to that of the real model, while allowing particles with comparable size to the real sand and controlling the simulated number of particles to limit computational cost.

- (1) For slopes with a high first bench, different failure modes were observed in the test and the simulation. This discrepancy may be attributed to simulated model's more homogeneous material properties.
- (2) Localized failure near the boundary in the experiments, observed in the experiment, did not occur in the simulation, suggesting that the compaction level near the walls in the experiment was lower than in other areas.
- (3) With the addition of a bench, the force chains extend further to reach the bench toe, providing additional resisting force to the slope and distributing the stress induced by external loading.
- (4) For the benched slope, at the onset of dynamic excitation, most force chains were concentrated in the slope area. As shaking continued, an increasing number of chains formed within the bench region. Under low-intensity dynamic excitation, force chains could be classified into main and subgrade chains. The main chains retained their orientation throughout the shaking, with their magnitude gradually increasing, while subgrade chains shifted direction periodically with the shaking. Sliding could occur when the main chains fractured. Under strong shaking, all force chains exhibited oscillations, and failure was more likely when their orientations aligned perpendicularly to the slope surface.

In **Chapter 9**, it was evaluated that the impact of shear pin alignments on benched slopes' dynamic stability. The installation of shear pins can enhance seismic resistance and alter the failure modes of benched slopes. Due to seismic amplification, the slope crest is the most vulnerable area and requires retrofitting. Shear pins installed in rows with narrow spacing on the surface of the upper bench can enhance the dynamic stability of the benched slope. Column alignment with small gaps can also improve stability to some extent, but it may still induce local sliding. Alignment by matrix with a large

gap between pins may weaken the slope integrity and reduce its stability. Therefore, the alignment of shear pins is more crucial than their quantity because the pins primarily bear shear force rather than tensile force under dynamic conditions.

Finally, future studies can be conducted in the following areas based on the limitations of this study:

- (1) The FS was determined solely based on force equilibrium through Culmann's method for ease of application in engineering practice. For a more rigorous theoretical approach, moment equilibrium should also be considered. Based on Taylor (1937)' chart, the closed-form solution based on moment equilibrium will be derived in the future.
- (2) The theories derived in **Chapter 5** and **Chapter 7** primarily focus on simplified benched slopes with only two benches. However, in engineering projects, multi-stage benched slopes are more common. Therefore, the analytical model should be extended to address these more prevalent configurations, and corresponding simulations should also be conducted.
- (3) In **Chapter 6** and **Chapter 7**, the pseudo-static method was utilized. However, this approach simplifies dynamic forces into a single constant horizontal or vertical force, neglecting the time-dependent and cyclic characteristics of seismic loading. It also fails to consider the amplification of seismic waves within the slope, which can have a significant impact on stress distribution and failure mechanisms. In this thesis, the peak value of acceleration was determined by taking the maximum absolute value recorded at the bottom of the model. Due to seismic amplification, this method yielded conservative results, as the entire slope experienced higher magnitudes of seismic excitation. Therefore, alternative methods for determining the k_h , more advanced approaches, such as the Newmark method or pseudo-dynamic method, should be considered for future studies.
- (4) In **Chapter 8**, simulation results provided explanations for certain phenomena observed in centrifugal modeling. Factors such as water content homogeneity, centrifugal loading, and compaction degree may significantly influence test results. However, these assumptions have not yet been verified. Thus, further DEM simulations are recommended to reproduce these phenomena and investigate the relationship between benched slope stability and these factors. Additionally, contact parameters were obtained from DST, which may overlook their variation during seismic loading. To improve calibration, cyclic DST should be conducted.
- (5) In **Chapter 9**, the optimal configurations of shear pins were identified through dynamic centrifugal tests, and their retrofitting mechanism was briefly explained. Further theoretical development is needed to clarify the relationship between shear pin alignment and resisting force. Additionally, DEM simulations should be performed for verification, as they offer a more effective approach to investigating the performance of shear pins compared to other methods. This is because the contact model in DEM requires fewer assumptions, providing greater accuracy in capturing the interaction mechanisms.
- (6) In real engineering applications, benched slopes often bear external loads from buildings, vehicles, or other structures. The interaction between these artificial loads and slope safety should be

investigated to ensure reliable design.

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